Page

7.0 Structural Steel 7.1-1
7.0.1 Introduction 1
7.1 Design Considerations 1
7.1.1 General 1
7.1.2 Girder Depth 1
7.1.3 Girder Spacing 2
7.1.4 Estimating Structural Steel Weights 2
7.1.5 Types of Steel 2
7.1.6 Available Plate Sizes 3
7.1.7 Girder Segment Sizes 3
7.1.8 Computer Programs 3
7.1.9 Fasteners 4
7.2 Girder Bridges 7.2 1
7.2.1 General 1
7.2.2 “I” Girders 1
7.3 Design “I” Girders 7.3-1
7.3.1 General 1
7.3.2 Composite Section 1
7.3.3 Flanges 1
7.3.4 Webs 1
7.3.5 Transverse Intermediate Stiffeners 1
7.3.6 Longitudinal Stiffeners 2
7.3.7 Bearing Stiffeners 2
7.3.8 Crossframes 2
7.3.9 Bottom Laterals 3
7.3.10 Bolted Field Splice 5
7.3.11 Camber 5
7.3.12 Roadway Slab Placement Sequence 7
7.3.13 Bridge Bearings 7
7.3.14 Surface Roughness 7
7.3.15 Welding 9
7.3.16 Fabrication 10
7.4 Plan Details 7.4-1
7.4.1 General 1
7.4.2 Structural Steel Notes 1
7.4.3 Framing Plan 1
7.4.4 Girder Elevation 1
7.4.5 Typical Girder Details 1
7.4.6 Crossframe Details 2
7.4.7 Camber Curve and Bearing Stiffener Details 2

7.4.8 Roadway Slab 2
7.4.9 Safety Cable Details 2
7.5 Shop Plan Review 7.5-1
7.99 Bibliography 7.99-1

Appendix A

7.0-A1 Steel Plate Girder Design Flow Chart
7.4-A1 Girder Framing Plan and Elevation View
7.4-A2 Part Longitudinal Girder Elevation
7.4-A3 Primary Stiffeners
7.4-A4 Transverse Intermediate Stiffeners
7.4-A5 Splices
7.4-A6 Optional Web Splices
7.4-A7 Fillet Weld Termination Detail
7.4-A8 Field Splice Detail
7.4-A9 Drip Plate Details
7.4-A10 Crossframes
7.4-A11 Crossframe Attachment Details
7.4-A12 Lateral Plate Detail
7.4-A13 Camber Curve and Bearing Stiffener Camber Details

7.0 Structural Steel

7.0.1 Introduction

The most common type of steel superstructure used on bridges in Washington State is the built-up steel "I" girder. Rolled beams have been used on a very limited basis but much of the following is applicable. Because of their uniqueness and limited application, other types of steel superstructures (box girders, trusses, arches, suspension, etc.) are not addressed.

Use English units for all widening and rehabilitation on existing English designed and detailed steel bridge projects. Metric units are acceptable for new previously designed steel bridge projects.

P65:DP/BDM7

7.1 Design Considerations

7.1.1 General

Use the Strength Design Method Load Factor Design of Section 10 Structural Steel of AASHTO Standard Specifications for Highway Bridges to design steel girders. Bridges on horizontal curves shall also meet the requirements of the AASHTO Guide Specifications for Horizontally Curved Highway Bridges, as applicable. The information provided in this chapter is intended to help apply these AASHTO specifications and to define office practice.

Typical construction is nonshored steel girders acting compositely with a reinforced concrete roadway slab. This is discussed in more detail in Section 7.3.2. Since plate stock of M 270 grade 50W and M 270 grade 50 are close in price, office practice is to specify grade 50W for plate girders.

The use of nonredundant load path structures should be avoided. Nonredundant loadpath structures are structures where the failure of a single load carrying member, or a component thereof, could cause a total collapse. An example would be a twin plate girder structure.

Nonredundant structures are generally not used because of the extensive ongoing annual maintenance inspections required by FHWA. Also, nonredundant structures increase fabrication costs and require greater attention to detail during design. Even so, the use of nonredundant structures may be approved by the Bridge Design Engineer, however, approval shall be obtained by the designer prior to beginning the design.

Steel girder bridges typically require a paint system to provide protection against corrosion. The paint system for girder bridges is defined in the Special Provisions and is a three-part system. The first coat is an inorganic zinc shop primer. This is a sacrificial protection system. The second coat is an epoxy seal normally applied after the slab has been placed. This is a barrier protective system but in combination with the zinc primer, is considered a composite protective system. The third and final coat is a urethane which protects the epoxy from UV attack and provides color for the bridge. The color is specified in the Special Provisions. This paint system will normally require repainting in approximately 30 years.

Unpainted weathering steel should be considered for locations deemed appropriate. See NCHRP Report 314. Approval for its use must be obtained from the Bridge Design Engineer. Careful attention to details is required for proper weathering. Accumulation of debris, staining of substructure, and water from expansion joints can pose considerable problems and add to life cycle costs. Provisions to sand blast erected steel and apply controlled wet-dry cycles are required to produce a sound protective coating with good appearance. Recommendations for using weathering steel are contained in Uncoated Weathering Steel Bridges, Vol. I, Chapter 9 of AISC's Highway Structures Design Handbook. A more comprehensive treatment is found in NCHRP Report 314 Guidelines for the Use of Weathering Steel in Bridges. Surfaces to be embedded in concrete, such as top flanges, should be shop painted.

7.1.2 Girder Depth

The superstructure depth is initially determined during preliminary plan development and is based upon the span/depth ratios provided in Chapter 2 of this manual. The designer will have to verify this depth by meeting live load deflection requirements and by meeting stress requirements. It is office practice to limit live load deflections to L/800 for HS-25 or L/1000 for HS-20. Live load deflection is calculated on a per bridge basis with reduction for multiple lanes.

The superstructure depth is typically the distance from the top of the concrete roadway slab to the bottom of the web. This distance is in multiples of 6 inches for shorter span bridges, and 1 foot 0 inches for longer span bridges, and should be consistent throughout the length of the bridge.

Other features such as notching at hinges (combined with notching for expansion joint system), vertical clearances, etc., should be considered in selecting the superstructure depth.

7.1.3 Girder Spacing

For simplicity of design, girders should be spaced such that each is designed for the same load; that is basically, girders will be identical. Spacing should be such that slab dead load is equally distributed on all girders and the distribution of wheel loads on the exterior girder is close to that of the interior girder. Barrier weights shall be equally distributed to a maximum of two "I" girders. The least number of girders should be used that is consistent with a reasonable deck design.

In general, live load distribution to girders shall be in accordance with AASHTO Section 3, Part C for "I" girders. When these bounds are exceeded, a rational live load distribution method should be used.

7.1.4 Estimating Structural Steel Weights

For the preliminary quantities or preliminary girder design, an estimate of steel weights for built-up plate composite "I" girders can be obtained from Figures 7.1.4-1 through 7.1.4-3. These figures are based upon previous designs with HS-20 live loads with no distinction between service load designs and load factor designs. These charts provide a good double check on final quantities.

The weights shown include webs, flanges, and all secondary members (web stiffeners, diaphragms, crossframe, lateral systems, gusset plates) plus a small allowance (usually 5 percent or less) for weld metal, bolts, and shear connectors.

7.1.5 Types of Steel

The most common types of steel used for bridges are now grouped in ASTM A 709 or AASHTO M 270 specifications. The following table shows equivalent designations. Grades of steel are based on minimum yield points.

ASTM	ASTM A 709	AASHTO	AASHTO M 270
A 36	Grade 36	M 183	Grade 36
A 572 gr 50	Grade 50	M 223 gr 50	Grade 50
A 588	Grade 50W	M 222	Grade 50W
A 852	Grade 70W	M 313	Grade 70W
A 514	Grade 100	M 244	Grade 100
	Grade 100W		Grade 100W

Plates and rolled sections are available is these specifications and grades. Rolled sections include beams (W, S, and M shapes), H-piles, tees, channels, and angles. These materials are prequalified under the Bridge Welding Code.

Use AASHTO M 270 grade 50W for plate girders. The fabricated costs of structural carbon and structural low alloy steel plate girders are about equal. The use of M 270 grade 100, 100W requires approval by the Bridge Design Engineer.

Availability of weathering steel can be a problem for some sections. For example, steel suppliers do not stock angles or channels in weathering steel. Weathering steel wide flange and tee sections are available but difficult to locate. Also, AASHTO M 270 steels are not stocked by local suppliers. The use of M 270 steel should be restricted to large quantities such as found in typical plate girder projects.

Structural tubes and pipes are covered by other specifications. See Table 1-4 of AISC *Manual of Steel Construction* for selection and availability. These materials are not considered prequalified under the Bridge Welding Code. They are covered under the Structural Welding Code AWS D1.1. Structural tubing ASTM A 500 is not recommended for dynamic loading applications.

7.1.6 Available Plate Sizes

Readily available lengths and thicknesses of steel plates should be used to minimize costs. Tables of standard plate sizes have been published by various steel mills and should be used for guidance. These tables are available through the steel specialist.

In general, an individual plate should not exceed 14 feet in width, including camber requirements, or a length of about 60 feet. If either or both of these dimensions are exceeded, a butt splice is required and should be shown or specified on the plans.

Plate thicknesses of less than $^{5}/_{16}$ inches should not be used for bridge applications.

7.1.7 Girder Segment Sizes

Locate bolted field splices so that individual girder segments can be handled, shipped, and erected without imposing unreasonable requirements on the contractor. Crane limitations need to be considered in congested areas near traffic or buildings. Transportation route options between the girder fabricator and the bridge site can effect the size and weight of girder sections allowed. The region should help determine the possible routes, and the restrictions they impose, during preliminary planning or early in the design phase.

"I" girder segment lengths should be limited to 150 feet depending upon their cross section. Weight is seldom a controlling factor. However, 40 tons is a practical limit for some fabricators. Long, slender segments can be difficult to handle and ship due to their flexibility. Horizontal curvature of girder segments may increase handling and shipping concerns.

Consider the structure's span length and the above factors when determining girder segment lengths.

7.1.8 Computer Programs

The designer should consult the design supervisor to determine the computer program currently being used for analyses. Instruction manuals for the programs are available in the Bridge Office Computer Section.

Office practice and good engineering principles require that the results of any computer program should be independently verified for accuracy. Verification is necessary to identify input errors which renders erroneous output. Also, programs with built-in code checks must be checked for default settings. Default settings may reflect old code or office practice may supercede the code that the program was written for.

7.1.9 Fasteners

All bolted connections shall be friction type. Design is based on Class B coating on fraying surfaces. The term "slip critical" implies a friction type connection.

Properties of High-Strength Bolts

Material	Bolt Diameter	Tensile Strength ksi	Yield Strength ksi
AASHTO M 164 (ASTM A325)	1/2 - 1″ inc.	120	92
	1 1/8 - 1″ inc.	105	81
	Over 1 1/2″	Not Available	
ASTM A 449 (No AASHTO equivalent)	1/4″ - 1″ inc.	120	92
	1 1/8 - 1 1/2″ inc.	105	81
	1 3/4″ -3″ inc.	90	58
	Over 3″	Not Available	
AASHTO M 314 ASTM F 1554 Grade 105	1/4″ - 3″ inc.	125-150	105
AASHTO M253 (ASTM A 490)	1/2″ - 1 1/2″ inc.	150-170	130
	Over 1 1/2″	Not Available	
ASTM A 354 Grade BD (No AASHTO equivalent)	1/2 - 2 1/2″ inc.	150	130
	3″ - 4″ inc.	140	115
	Over 4″	Not Available	

General Guidelines for Steel Bolts

1. M 164 (A325) — High strength, headed structural steel bolts for use in structural joints. Suitable heavy hex nuts and plain hardened washers are covered by this specification. These bolts may be hot-dip galvanized. Do not specify for anchor bolts.

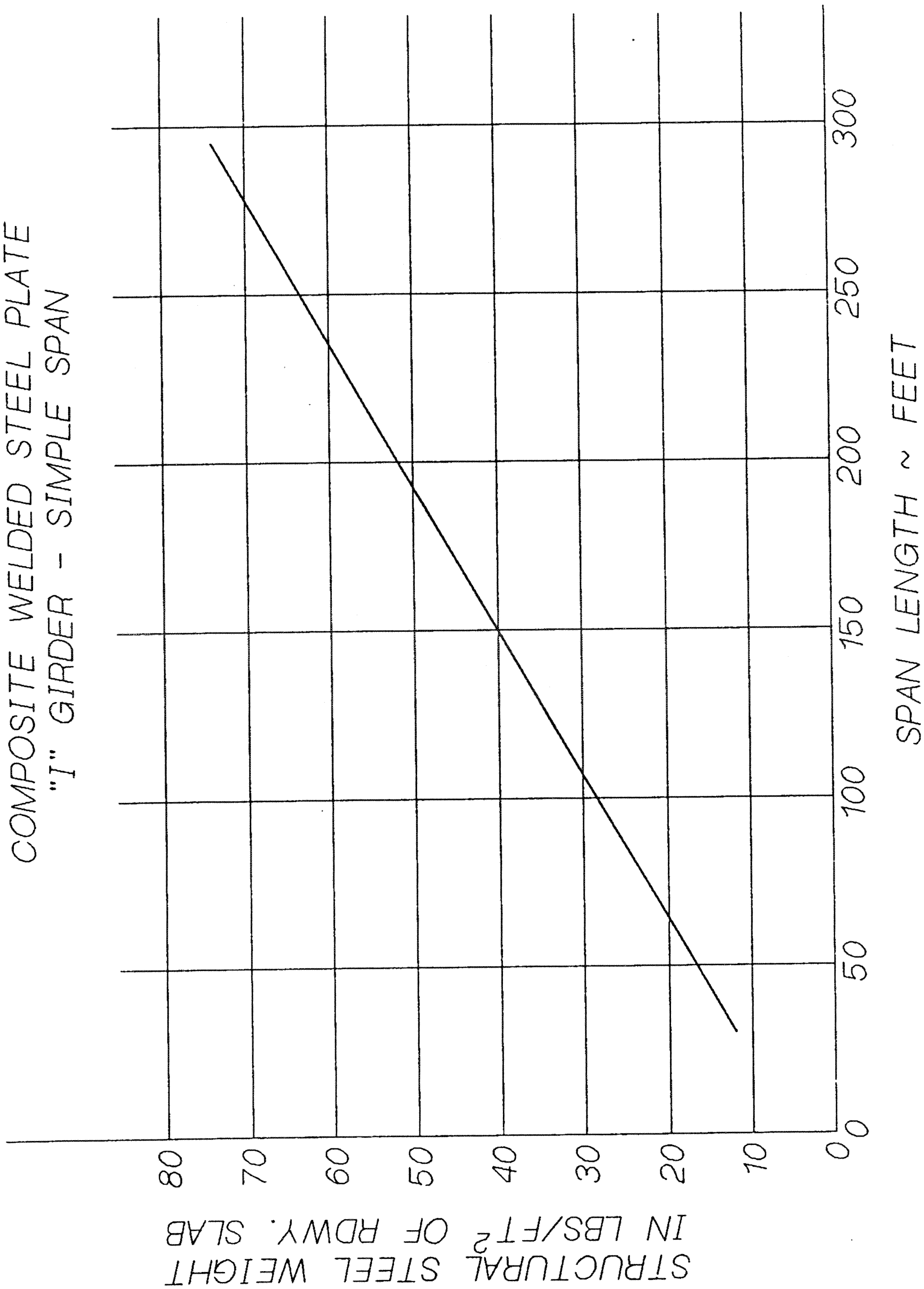
2. A449 — High strength steel bolts and studs for general applications including anchor bolts. Recommended for use as anchor bolts where strengths equivalent to A325 bolts are desired. These bolts may be hot-dip galvanized.

3. M 314 (F1554) — Grade 105 — Higher strength anchor bolts to be used for larger sizes (1½″ to 3″). These bolts are not covered in the *Standard Specifications* so they require coverage in the Special Provisions when called for.

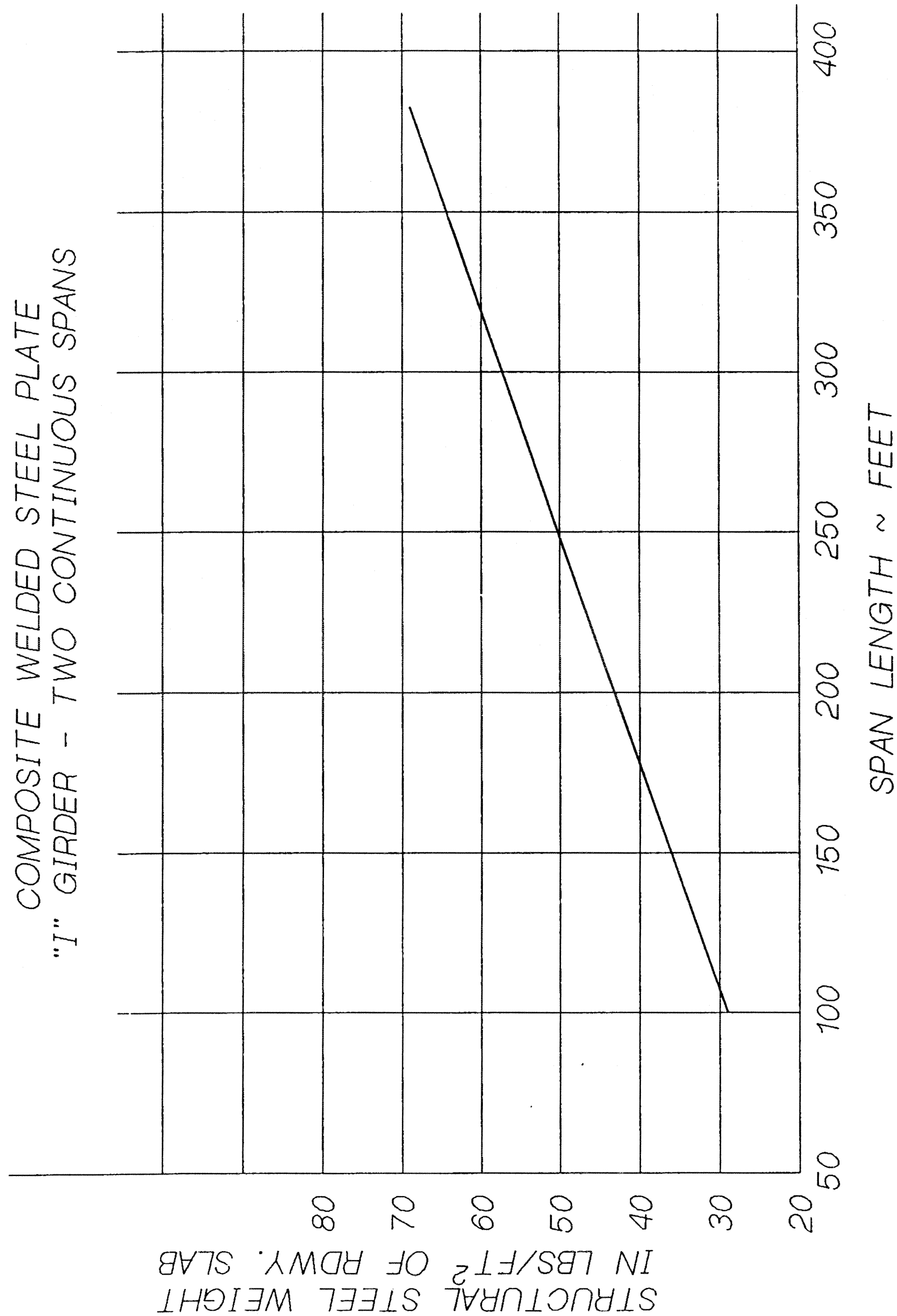
4. M 253 (A490) — High strength alloy steel headed bolts for use in structural joints. These bolts should not be galvanized, because of the high susceptibility to hydrogen embrittlement. In lieu of galvanizing, the application of two or three coats of an approved zinc rich paint may be specified. Suitable heavy hex nuts and plain hardened washers are covered by this specification. Do not specify for anchor bolts.

5. A354 — Grade BD — high strength alloy steel bolts and studs. These bolts are suitable for use as anchor bolts where strengths equal to A490 bolts are desired. Nuts and washers are covered by this specification.

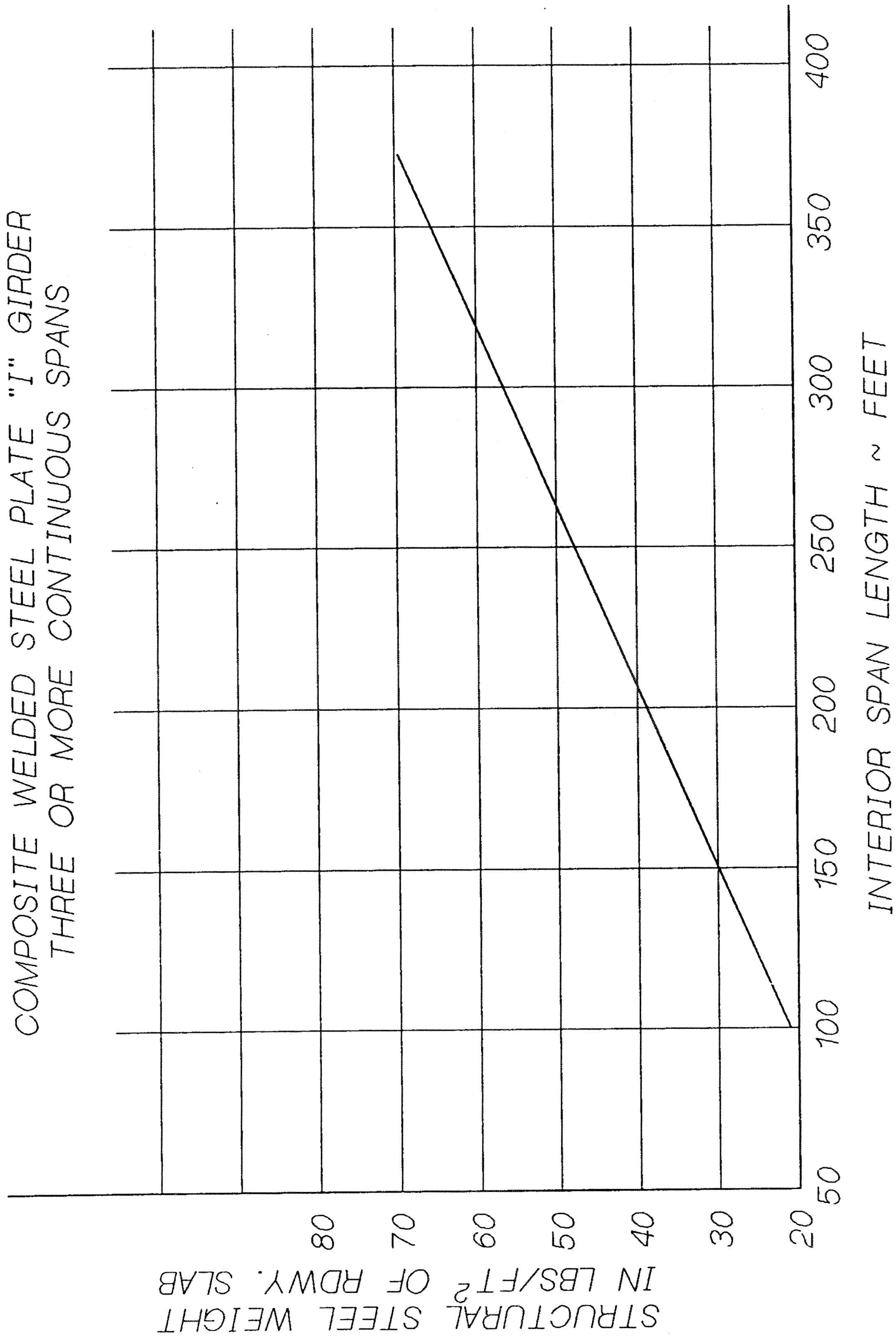
   These bolts should be treated in the same manner as A490 bolts in regard to galvanizing.



Composite Welded Steel Plate "I" Girder — Simple Span
Figure 7.1.4-1



Composite Welded Steel Plate "I" Girder — Two Continuous Spans
Figure 7.1.4-2



Composite Welded Steel Plate "I" Girder — Three or More Continuous Spans
Figure 7.1.4-3

Equivalent ASTM and AASHTO Specifications

ASTM Designations	AASHTO Designations
A 6/A 6M	M 160/M 160M
A 27/A 27M	M 103/M 103M
A 36/A 36M	M 183/M 183M
A 48	M 105
A 53	No Equivalent
A 108	M 169
A 109	No Equivalent
A 109M	No Equivalent
A 123	M 111
A 153	M 232
A 252	No Equivalent
A 307	No Equivalent
A 325	M 164
A 325M	M 164M
A 328/A 328M	M 202/M 202M
A 354	No Equivalent
A 370	T 244
A 435/A 435M	No Equivalent
A 446/A 446M	No Equivalent
A 449	No Equivalent
A 486/A 486M	M 192/M 192M
A 490	M 253
A 490M	M 253M
A 500	No Equivalent
A 501	No Equivalent
A 502	No Equivalent
A 514/A 514M	M 244/M 244M
A 525	No Equivalent
A 525M	No Equivalent
A 536	No Equivalent
A 563	M 291
A 563M	M 291M
A 572/A 572M	M 223/M 223M
A 588/A 588M	M 222/M 222M
A 618	No Equivalent
A 668	M 102
A 673/A 673M	T 243/T 243M
A 709/A 709M	M 270/M 270M
A 852/A 852M	M 313/M 313M
A 898/A 898M	No Equivalent
B 695	M 298
F436	M 293
F436M	No Equivalent
F606	No Equivalent
F 606M	No Equivalent
F 959M	No Equivalent
F 1554	M 314

Figure 7.1.5-1

7.2 Girder Bridges

7.2.1 General

Once the material of choice, steel has been eclipsed by concrete. Numerous graphs and charts are available to demonstrate the falling percentage of steel bridges and the rising percentage of concrete bridges being constructed. Corrosion and fatigue cracking have contributed to unanticipated life cycle costs. Fabrication and material costs have also contributed to steel's relative cost disadvantage. These trends may be compensated for by simplification of fabrication details, elimination of expansion joints and hinges, and the lowering of steel prices due to the advent of mills that recycle scrap iron.

The specifications allow a combination of plastic design in positive moment regions and elastic design in negative moment regions. Plate girders, of the depths typically built in this state, have traditionally been designed to elastic limits or lower. Newer design methods may help reduce steel weight and narrow the cost gap between steel and concrete bridges. Steel girders can also be shallower than the same span prestressed girders.

7.2.2 "I" Girders

As stated in the introduction, welded plate "I" girders constitute the majority of steel girders designed by WSDOT. The "I" girder represents an efficient use of material for maximizing stiffness. Its shortcoming is inefficiency in resisting shear. Office practice is to maintain constant web thickness for short to medium span girders. Weight savings is achieved by minimizing the number of webs used for a given bridge. This also helps minimize fabrication, handling, and painting costs. Current office practice is to use a minimum of three girders to provide redundant load path structures. Two girder superstructures are considered non-redundant and hence, fracture critical.

Steel plate girder design is complicated by buckling behavior of relatively thin elements. Most strength calculations involve buckling in some form. Either a minimum thickness condition must be met to achieve a given stress state, or strength is reduced by some amount to account for buckling. Buckling can be a problem in flanges as well as webs if compression is present. Also, stability needs to be insured for all stages of construction, with or without a roadway deck. The art of designing these girders is to minimize material and fabrication expense while ensuring adequate strength and stiffness.

"I" girders are an excellent shape for welding. All welds for the main components are easily accessible and visible for welding and inspection. The plates are oriented in line with the rolling direction so as to make good use of strength, ductility, and toughness of the structural steel. The web is attached to the top and bottom flanges with continuous fillet welds. Usually, they are made with automatic submerged arc welders. These welds are loaded parallel to the longitudinal axis and resist horizontal shear between the flanges and web. Minimum size welds based on plate thickness controls design in most cases. The flanges and webs are fabricated to full segment length with full penetration groove welds. These welds are inspected by ultrasound (UT) 100 percent. Tension welds, as designated in the plans, are also radio-graphed (RT) 100 percent. Office practice is to have flanges and webs fabricated full length before they are welded into the "I" shape. Weld splicing built-up sections results in poor fatigue strength and zones that are difficult or impossible to inspect.

7.3 Design "I" Girders

7.3.1 General

Composite girders may be used for continuous and simple spans. As mentioned in Section 7.1.1, office practice is to use nonshored girders. The girder section must carry the weight of the fluid (wet) concrete deck as well as its own dead load. After the concrete has cured, the composite section becomes effective in carrying all superimposed loads. Shear connectors are provided over the full length of the top flange of the structure or continuous portions of the structure. The stiffness analysis is performed for superimposed dead loads and live load plus impact, assuming the section acts compositely over the total length of the structure or continuous portion thereof.

The fatigue truck shall be HS-20 for LFD design. When designing by the LRFD method, the fatigue truck shall be applied without neglecting axles that do not contribute to the extreme force effect. Assume Case I road type when determining the number of stress cycles for design.

7.3.2 Composite Section

Short-term primary loading live load plus impact is applied to the composite section transformed using ES/EC, commonly denoted n. Long-term loading (dead load of barriers, signs, luminaries, overlays, etc.) is applied to the composite section transformed using 3 ES/EC. The moments resulting from the stiffness analysis are applied to the composite section in the positive moment region.

The negative moments from the analysis are applied to the steel girder section including longitudinal reinforcing (negative moment composite section).

Longitudinal reinforcing steel shall be used in negative moment regions of composite, continuous spans. Refer to AASHTO Section 10.38.4.3.

7.3.3 Flanges

When determining girder section at locations of maximum positive and negative moment, try to use a constant top and bottom flange width throughout the length of the bridge. If a width change in the top flange is necessary, it is best made at a field splice. The cross sectional areas of the top and bottom flanges may be varied by changing thickness. Generally two changes in girder section located within the negative moment region, one each side of maximum moment and between field splices, will be most economical. Flange thickness changes at field splices are easily accomplished. One girder section change in end spans between maximum positive moment and end bearing may be justified.

As a general rule, a welded splice may be justified if more than 500 pounds of steel can be saved.

7.3.4 Webs

Maintain constant web thickness throughout the structure. Except for extremely deep superstructures, maintain webs full depth without longitudinal splices. Vertical web splices for girders should be shown on the plans in an elevation view with additional splices made optional to the fabricator. All welded web splices on exterior faces of exterior girders and in tension zones elsewhere shall be ground smooth. Like splices on interior girders need not be ground in compression zones.

7.3.5 Transverse Intermediate Stiffeners

These stiffeners shall be used in pairs at crossframe locations on interior girders and on the inside of webs of exterior girders. They shall be welded to the top flange, bottom flange and web at these locations. This detail is considered fatigue stress category C. Stiffeners used between crossframes shall be located on one side of the web, welded to the compression flange, and cut short of the tension flange. Stiffeners located between crossframes in regions of stress reversal shall be welded to one side of the web and cut short of both flanges. Alternatively, they may be welded to both flanges if fatigue Category C is checked.

7.3.6 Longitudinal Stiffeners

On long spans where web depths exceed 12 feet, comparative web evaluations shall be made to determine whether the use of longitudinal stiffeners will be more economical. Fabrication costs indicate the use of longitudinal stiffeners is not economical on webs 12 feet deep or less.

7.3.7 Bearing Stiffeners

Stiffeners are required at all bearings to enable the reaction to be transmitted from the web to the bearing. These stiffeners are designated as columns, therefore, must be vertical under total dead load. The connection of the bearing stiffener to the girder consists of full penetration groove welds to the bottom flange and fillet welding to the top flange and web. These connection details limit the design stress to Category C for all girder sections at points of maximum negative moment.

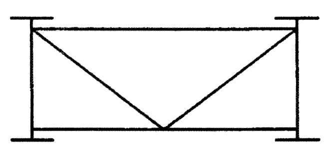
In the case of severe horizontal curvature on structures where girders and crossframes are subjected to large transverse forces resulting in considerable lateral flange bending, full penetration welds at top and bottom flanges may be necessary. Full penetration welds are expensive and should be used only where necessary.

7.3.8 Crossframes

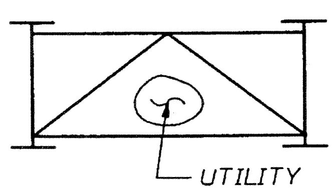
The primary function of intermediate crossframes is to distribute vertical loads transversely and give torsional rigidity to the superstructure. Together with the bottom laterals they stabilize the superstructure during erection, pouring, and curing of the roadway slab. On curved bridges, the crossframes also resist lateral flange bending. Pier crossframes are subjected to lateral loads that originate primarily from wind, earthquake, and curvature and are transmitted from the roadway slab to the bearings.

Crossframes are generally patterned as K-frames or as X-frames. Typically the configuration selected is based on the most efficient geometry. The members should closely approach a slope of 1:1 or 45°. Avoid conflicts with utilities passing through the girders.

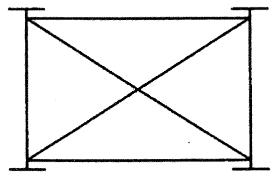
On K-frames like the following, avoid connection congestion at bottom laterals:



K-frames like the following may be better for utilities, however, create some congestion at the bottom lateral connection:



X-frames like the following, where girder depth approaches girder spacing, are more efficient geometrically:



Intermediate crossframes for straight girders with little or no skew should be designed as secondary members. Choose a section which satisfies $\frac{kL}{r} \leq 140$ and design connections only for anticipated loads, not for 75 percent strength of member. This should result in greater economy but still meet the intent of AASHTO specifications.

In general, crossframes should be installed parallel to piers for skew angles of 0° to 10°. For greater skew angles, other arrangements may be used. Consult with the design unit supervisor or the steel specialist for special requirements.

Intermediate crossframes for curved I-girders require special consideration. Curved girder systems should be designed according to AASHTO "Guide Specifications for Horizontally Curved Highway Bridges." Use Table 1.4A of the guide specifications to distinguish between straight and curved girders.

Crossframes at piers must be designed to transmit transverse loads due to wind or earthquake from the roadway slab to the bearings or transverse stops. Design and detail pier crossframes separately from intermediate crossframes.

Bolted connections for crossframes are favored because they allow adjustment during fit up and erection.

Connections of crossframes to web stiffeners require careful attention to detailing to minimize fabrication difficulties and most importantly increase fatigue resistance. Web stiffeners at crossframes shall be welded to top and bottom flanges. This practice minimizes out-of-plane bending of the girder web. The resulting detail must be checked for Category C stress range.

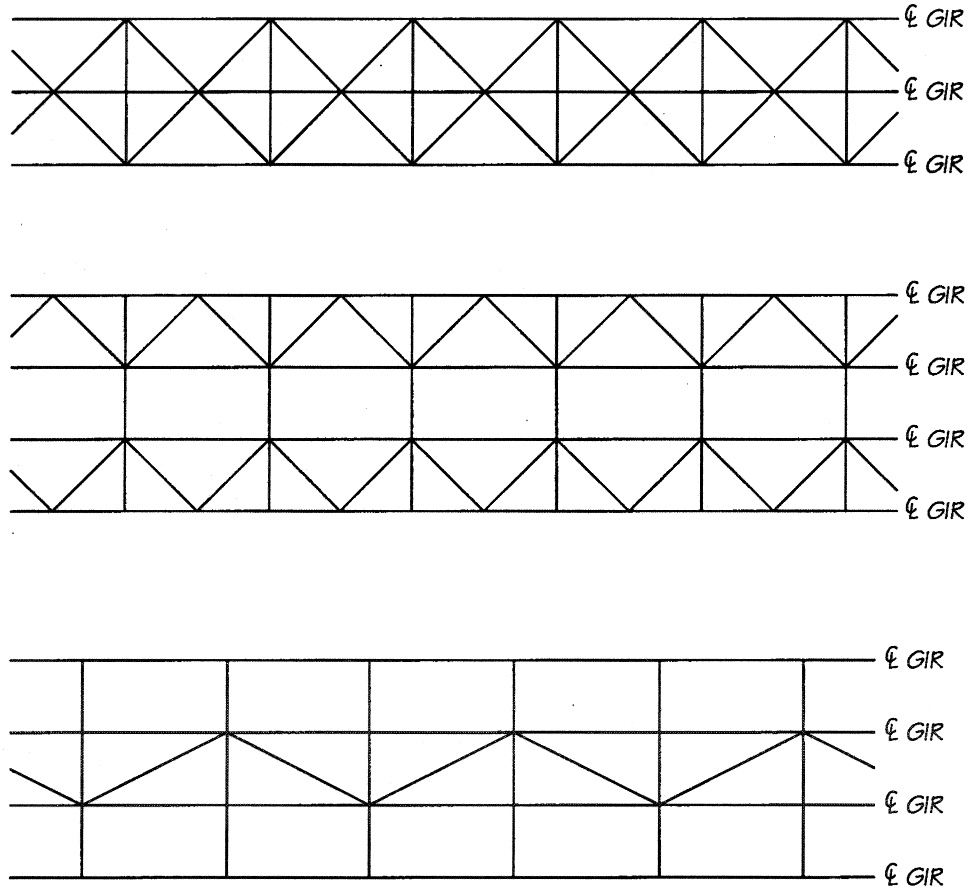
7.3.9 Bottom Laterals

The primary function of a bottom lateral system is to stabilize the girders against lateral loads before the deck hardens and stabilize the steel portion of the superstructure while the roadway slab is placed.

On straight bridges, office practice is to design the diagonal members in bottom laterals as secondary members. X-framing may be designed in tension only. K-framing must be designed as compression and tension members. One hundred fifty percent of the allowable service load design stress is permitted in the laterals for the temporary construction condition. Consult AASHTO for further guidance. Determine one size of diagonal member to be used throughout the structure. Partial loading (total panels less one-half of the end panel) yields maximum shear in the end panel.

Also, on curved structures, the bottom laterals are effective in resisting live load plus impact thereby becoming primary members and must be modeled in the structure to determine the actual forces the members experience.

Lateral patterns are formed depending on number of girder lines, girder spacing, and crossframe spacing. Cost considerations should include geometry, repetition, number, and size of connections. See Figure 5.1.2-1. Consideration should be given to limiting bottom laterals to one or two bays on straight bridges.



Examples of Lateral Bracing
Figure 7.3.9-1

Note: Where lateral gusset plates are welded to girder webs, the design stress level in the girder, at the web, is governed by the Category E detail.

For widening projects, bottom laterals are not needed since new can be braced against existing construction. Framing which is adequately braced should not require bottom laterals.

7.3.10 Bolted Field Splice

Office practice is to use bolted field splices. Splices are usually located at the dead load inflection point to minimize the design bending moment. The latest USS Highway Structures Design Handbook should be consulted for examples of splice designs. See AASHTO Section 10.18 for splice design requirements.

Splices should be designed for the greater of:

1. 75 percent of the moment capacity of the smaller section.
2. The average of the required moment due to factored loads and the moment capacity of the smaller section.

Web splice bolts are designed to resist a shear force due to:

1. Total factored shear force plus;
2. Shear force due to moment resulting from the above shear force times the eccentricity of the distances from the centerline of the splice to the center of gravity of the bolt group on one side of the centerline of the splice plus;
3. Shear force due to the portion of the design moment resisted by the web, which is:

$$\left(\frac{I_{WEB}}{I_{SECTION}}\right) \times \text{design moment at centerline of splice}$$

The outer most bolt in the bolt group is the most highly stressed. The shear force can be determined by using the "elastic moment of inertia" method.

The flange splice is designed to resist the portion of the design moment not resisted by the web.

Split splice plates are used at the bottom of the bottom flange to allow moisture to pass through the splice.

Fill plates are used to maintain constant flange splice plate thickness across the splice.

Allow fabricators to use steel sheet (ASTM A 715) for fill plates less than ¼ inch thick.

Fill plates are not subject to tension and therefore a charpy V-notch toughness test should not be required for them. Mark splice plates that carry tensile stress.

Allow fill plates to be fabricated from AASHTO M183, if steel is painted.

7.3.11 Camber

Permanent girder deflections shall be shown in the contract plans in the form of camber diagrams. The traditional format for detailing these diagrams should be adhered to for the benefit of construction. Camber curves are used by shop plan detailers, girder fabricators at the shop assembly stage, girder erectors, and field personnel. Most, if not all, phases of girder fabrication and erection involve potential sources of error in camber. Also, the Standard Specifications provide for adjustments at the time of slab forming. Therefore, the slab design should reflect the possibility of reduced slab depths.

Girder camber is accomplished at three stages of construction. First, girder webs are cut from plates so that the completed girder segment will assume the shape of camber superimposed on profile grade. The fabricated girder segment will incorporate the as-cut web shape and some degree of welding distortion. Next, the girder segments are brought together for shop assembly. Field splices are drilled as the segments are placed in position to fit profile grade plus total dead load camber. Finally, the segments are erected, sometimes with supports at field splices. There may be slight angle changes at field splices, resulting in altered girder profiles. Errors at mid-span can be between one to two inches at this stage.

The following is a general outline for calculating camber and is based on girders having shear studs the full length of the bridge.

Two curves will be required, one for total dead load plus slab shrinkage and one for girder self-weight (steel only).

Girder dead load deflection is determined by using various computer programs. Girder self-weight is assumed to include the basic section plus stiffeners, crossframes, welds, shear studs, etc. These items may be accounted for by adding an appropriate percentage of basic section weight. Fifteen percent of total girder weight, distributed evenly along the bridge, should suffice. This loading is applied to the steel section only.

Total dead load camber shall consist of:

1. Steel weight.
2. Slab weight.
3. Traffic barriers and overlays.
4. Slab shrinkage.

Slab dead load deflection will require the designer to exercise some judgment concerning degree of analysis. A two-span bridge of regular proportions, for example, should not require a rigorous analysis. The slab may be assumed to act instantaneously on the steel section only. Therefore, the calculation would be performed as above. For long structures, unusual girder arrangements, and especially structures with hinges, an analysis coupled with a slab pour sequence may be justified. This would require an incremental analysis where previous slab pours are treated as composite sections and successive slab pours are added on noncomposite sections. Each slab pour requires a separate deflection analysis. The total effect of slab construction is the superposition of each slab pour. A note must accompany the camber diagram explaining the relation between camber and the slab pour sequence. The contractor should be required to submit a new camber diagram if a different slab pour sequence is proposed.

Traffic barriers, overlays, and other items constructed after the slab pour should be analyzed as if applied to a composite section full length of the bridge. The modulus of elasticity of the slab concrete should be reduced to one third of its short term value. For example, if $f'c = 5000$ psi, then use a value of $n = 21$.

Slab shrinkage has a varying degree of effect on superstructure deflections. Again, the designer must use some judgment in evaluating this effect on camber. Slab shrinkage should be the smallest portion of the total camber (approximately 20 percent).

In addition to girder deflections, show girder rotations at bearing stiffeners. This will allow shop plan detailers to compensate for rotations so that bearing stiffeners will be vertical in their final position.

Camber tolerance is governed by the Bridge Widening Code AWS D1.5. A note of clarification is added to the plan camber diagram: "For the purpose of measuring camber tolerance during shop assembly, assume top flanges are embedded in concrete without a designed haunch." This allows a high or low deviation from the theoretical curve. In the past, no negative camber tolerance was allowed.

7.3.12 Roadway Slab Placement Sequence

The roadway slab is placed in a prescribed sequence allowing the concrete in each sequence to shrink freely. This minimizes cracking of the slab due to shrinkage. Furthermore, placing the slab sequentially allows the contractor to place manageable volumes of concrete at a time.

For the first sequence, concrete is placed on the dead load positive moment region of end spans and in the positive moment regions of alternate interior spans.

For the second sequence, concrete is placed on the dead load positive moment region of the remaining spans after the concrete in the first sequence has attained a minimum specified tensile strength. Check tensile stresses in the first sequence slab pour due to the second sequence slab pour.

For the third sequence, concrete is placed on the dead load negative moment region over each interior pier. Generally, slab placement in negative moment regions does not cause cracking in previously placed concrete.

7.3.13 Bridge Bearings

Office practice and design criteria for bridge bearings can be found in Chapter 8 of this manual.

7.3.14 Surface Roughness

The standard measure of surface roughness is the microinch value. It is specified by the symbol xxx√ and shall be shown on the plans for all surfaces for which machining is required unless covered by the Standard Specifications or Special Provisions. When used, this symbol means that the average value of the depth of the surface grooves shall not exceed xxx millionths of an inch. The lower the number (xxx), the smoother the surface.

Following is a brief description of some finishes:

√ 500 A rough surface finish typical of "as rolled" sections. Suitable for surfaces that do not contact other parts and for bearing plates on sheet lead or grout.

√ 250 A fairly smooth surface. Suitable for connections and surfaces not in moving contact with other surfaces. This finish is typical of ground edges in tension zones of flanges.

√ 125 A fine machine finish resulting from careful machine work using high speeds and taking light cuts. It may be produced by all methods of direct machining under proper conditions. Suitable for steel to steel bearing or rotational surfaces including rockers and pins.

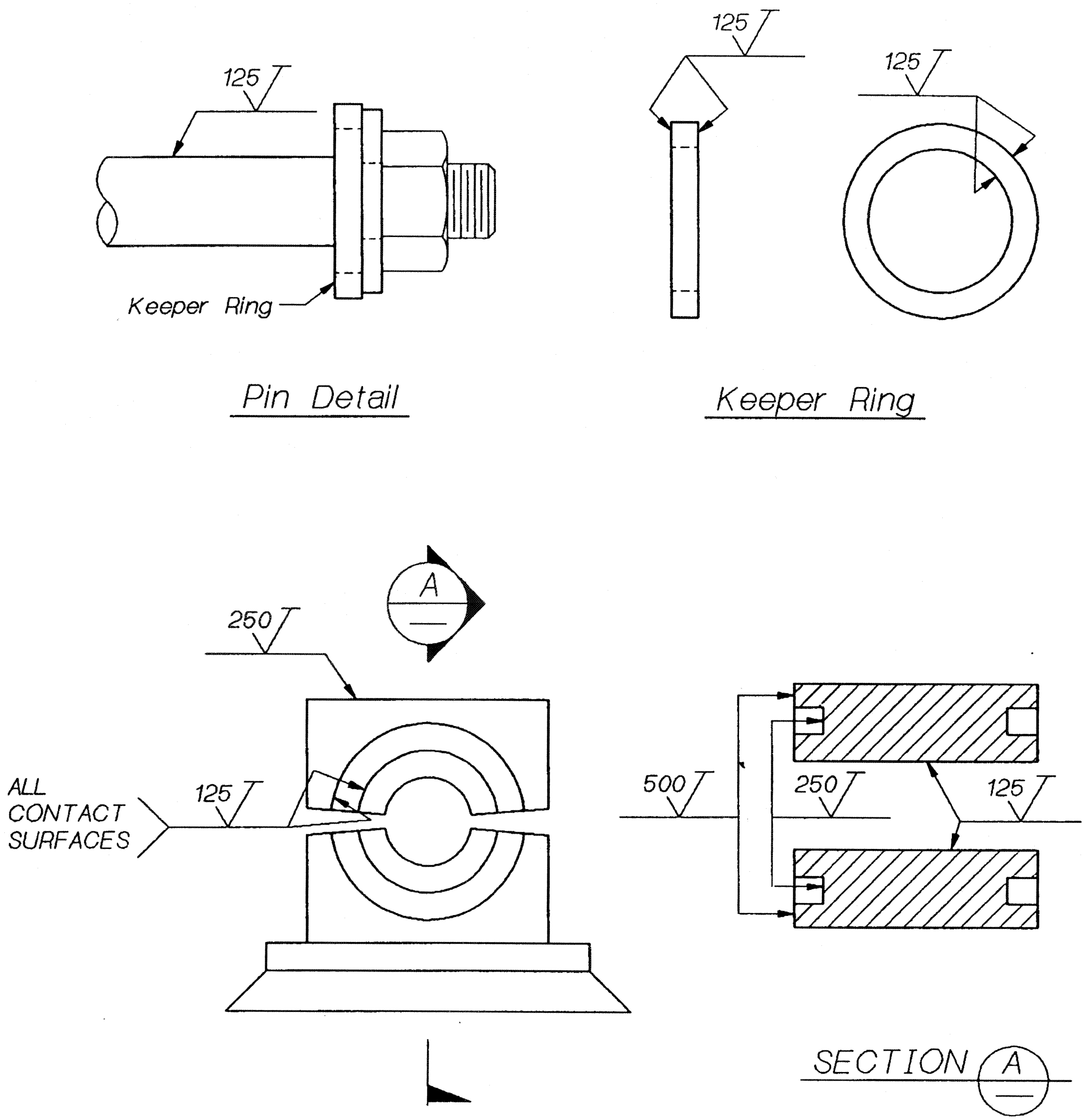
√ 63 A smooth machine finish suitable for high stress steel to steel bearing surfaces including roller bearings on bed plates.

√ 32 An extremely fine machine finish suitable for steel sliding parts. This surface is generally produced by grinding.

√ 16 A very smooth, very fine surface only used on high stress sliding bearings. This surface is generally produced by polishing.

For examples, see Figure 7.3.14.

For stainless steel sliding surfaces, specify a #8 mirror finish. This is a different method of measurement and reflects industry standards for polishing. No units are implied.



Surface Finish Examples
Figure 7.3.14

7.3.15 Welding

All structural steel and rebar welding shall be in accordance with the WSDOT Standard Specifications, amendments thereto and the special provisions. The Standard Specifications currently calls for welding structural steel according to the AASHTO/AWS D1.5-96 Bridge Welding Code (BWC) and the latest edition of the AWS D1.1 Structural Weld Code. The designers should be especially aware of current amendments to the following sections of the *Standard Specifications*, 6-03.3(25) Welding and Repair Welding and 6-03.3(25)A Welding Inspection.

Exceptions to both codes and additional requirements are shown in the Standard Specifications and the special provisions.

Standard symbols for welding, brazing, and nondestructive examination can be found in the ANSI/AWS A 2.4 by that name. This publication is a very good reference for definitions of abbreviations and acronyms related to welding.

The designer must consider the limits of allowable fatigue stress, specified for the various welds used to connect the main load carrying members of a steel structure. See Chapter 10 of AASHTO.

The minimum fillet weld size shall be as shown in the following table. Weld size is determined by the thicker of the two parts joined unless a larger size is required by calculated stress. The weld size need not exceed the thickness of the thinner part joined.

Base Metal Thickness of Thicker Part Joined Inches (mm)	Minimum Size of Fillet Weld Inches (mm)
To ¾ (20 mm) inclusive	¼ (6 mm)
Over ¾ (20 mm)	5/16 (8 mm)

The minimum size seal weld shall be 3/16 inch (5 mm) fillet weld.

In general, the maximum size fillet weld which may be made with a single pass is 5/16 inch for submerged arc, gas metal arc, and flux-cored arc welding processes. The maximum size fillet weld made in a single pass is ¼ inch for the shield metal arc welding process.

The major difference between AWS D1.1 and D1.5 is the welding process qualification. The only process deemed prequalified in D1.5 is shielded metal arc. All others must be qualified by test. Qualification of M 270 grade 50W (A709 grade 50W) in Section 5 of D1.5 qualifies the welding of all AASHTO approved steels with a minimum specified yield of 50 Ksi or less. Bridge fabricators generally qualify to M 270 grade 50W (A709 grade 50W).

All welding procedure specifications (WPS) submitted for approval must be accompanied by a procedure qualification record (PQR), a record of test specimens examination and approval except for SMAW prequalified. Some handy reference aids in checking WPS in addition to PQR are:

Matching filler metal requirements are found in BWC Section 4.

Prequalified joints are found in BWC Section 2.

AWS electrode specifications and classifications are obtained from the structural steel specialist.

Lincoln Electric Arc Welding Handbook.

Many of Lincoln Electric's published materials and literature are available through those designers and supervisors who have attended their seminars.

WSDOT *Standard Specifications* for preheat and Interpass temperatures.

Notes: Electrogas and electroslag welding processes are not allowed in WSDOT work. Narrow gap improved electroslag welding will be allowed on a case-by-case basis.

Often in the rehabilitation of existing steel structures, it is desirable to weld, in some form, to the inplace structural steel. Often it is not possible to determine from the contract documents for the structure whether or not the existing steel is weldable. WSDOT fabrication inspectors in the Northwest Region contract with a company which can make that determination economically. Coupons from the steel must be furnished for a spectrographic examination. Contact these inspectors to verify that the service is still available before making preparations.

7.3.16 Fabrication

In most cases, a one girder line progressive longitudinal shop assembly is sufficient to assure proper fit of subsections, field splices, and crossframe connections, etc., in the field. Due to geometric complexity of some structures, progressive transverse assembly, in combination with progressive longitudinal assembly may be desirable. The designer shall consult with the supervisor and the steel specialist to determine the extent of shop assembly and clarification of the *Standard Specifications*. The desired method of assembly shown in the *Standard Specifications* will then be required in the special provisions.

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7.4 Plan Details

7.4.1 General

Detailing practice should follow industry standards. Designations for structural steel can be found in Table 2-1 of AISC *Detailing for Steel Construction*. Old plans are a good reference for traditional detailing practices. Radical or even modest changes in detailing practice can result in misinterpretation of plans. Innovation is best reserved for content, not presentation of steel detailing.

Actual details for plate girders are continually being revised or improved. Cost benefits for individual details vary from shop to shop and even from time to time. For these reasons, previous plan details can be guides but should not be considered standards.

In general, office practice is to favor field bolted as opposed to field welded connections. In addition, members of cross frames are shop bolted to give some degree of field adjustment. Welded assemblies tend to be less adjustable when it comes time to install them.

7.4.2 Structural Steel Notes

Due to their dynamic nature, the structural steel notes are not shown in this manual. The designer's attention is directed to the Bridge and Structures Office Book of Knowledge (BOK) which contains the most current version of the structural steel notes in their entirety. These notes must be edited based on the requirements unique to each project and additions and deletions made accordingly.

7.4.3 Framing Plan

Define girders and component parts not shown on the girder elevation view such as jacking stiffeners. Locate panel points (crossframe locations). Show general arrangement of bottom laterals. Provide geometry, bearing lines, and transverse intermediate stiffener locations. Show field splices and detail the general configuration of crossframes in a section through framing plan.

For geometrically complicated structures, a rather detailed framing plan should be made to help guide the shop detailer and the shop plan reviewer.

7.4.4 Girder Elevation

Define flanges, webs, and components thereof. Show shear connector spacing, location, and number across the flange. Show shear connectors in the girder details also. Locate transverse intermediate stiffeners and show requirements for clearance from tension flange. Define those components of the girder subject to the Charpy V-notch requirements shown in the Standard Specifications. Define full penetration welds X or portions thereof subject to tension for which Radiographic (x-ray) examination is required. See Standard Specifications. V and X are mentioned also in the Structural Steel Notes, Section 7.4.2. Permissible welded web splices may show, however, the optional welded web splice shown elsewhere in the plans permits the fabricator to add splices subject to the approval of the engineer.

7.4.5 Typical Girder Details

One or two plan sheets should be devoted to showing typical details to be used throughout the girders. Such details include the weld details, various stiffener plates and weld connections, locations of optional web splices, and drip plate details. Include field splices here if only one type of splice will suffice for the plans. An entire sheet may be required for complicated bridges with multiple field splice designs. See Appendix 7.4-A1 to A9. Note: Do not distinguish between field bolts and shop bolts. A solid bolt symbol will suffice.

7.4.6 Crossframe Details

Typical crossframe and bottom lateral details are shown on Appendix 7.4-A10 to A12. Actual lengths of members and dimensions of connections will be determined by the shop plan detailer. Details should incorporate actual conditions such as skew and neighboring members so that geometric conflicts can be minimized. Tee sections are preferred over double angles for easier painting. If double angles are used, leave a minimum of 1 inch between legs and include fillers as needed for stability.

7.4.7 Camber Curve and Bearing Stiffener Details

Camber curves should be detailed using conventional practices. Dimensions given at tenth points has been office practice in the past. In lieu of tenth points, dimensions may also be given at crossframe locations which are more useful in the field. See Appendix 7.4-A13.

7.4.8 Roadway Slab

The roadway slab is detailed in section and plan views. For continuous spans, add a section showing negative moment longitudinal reinforcing to the typical section shown at mid-span. If possible, continue the positive moment region reinforcing pattern from end-to-end of the slab with the negative moment region reinforcing superimposed on it. The plan views should detail typical reinforcing and cutoff locations for negative moment steel. Avoid termination of all negative moment steel at one location. See Appendix 7.4-A14 and A15.

The "pad" dimension for steel girders is treated somewhat differently than for prestressed girders. The pad dimension is assumed to be constant throughout the span length. Ideally, the girder is cambered to compensate for dead loads and vertical curves. However, fabrication and erection tolerances result in considerable deviation from theoretical elevations. The pad dimension is therefore considered only a nominal value and is adjusted as needed along the span once the steel has been erected and profiled. The screed for the slab is to be set to produce correct roadway profile. The plans should reference this procedure contained in *Standard Specification* 6-03.3(39). The pad dimension is to be noted as nominal. As a general rule of thumb, use 11″ for short span rolled beam bridges, 12″ for short span plate girder bridges (150′ to 180′), 13″ for medium spans (180′ to 220′) and 14″ to 15″ for long spans (over 220′). These figures are only approximate. Use good engineering judgment when detailing this dimension.

7.4.9 Safety Cable Details

Safety cables for maintenance crews are standardized details. If room permits, include safety cables with typical girder details. Cable locations may be adjusted to avoid conflicts with other details such as large gusset plates. See Appendix 7.4-A16.

7.5 Shop Plan Review

Shop plans must be checked for agreement with the Contract Plans, *Standard Specifications*, and the special Provisions. The review procedure is described in Section 1.3.5 of this manual.

Welding procedure specifications and procedure qualification records should be submitted with shop plans. If not, they should be requested and received before shop plans are approved.

Most shop plans may be stamped:

> "GEOMETRY NOT REVIEWED
> BY THE BRIDGE & STRUCTURES OFFICE"

However, the reviewer should verify that lengths, radii, and sizes shown on shop plans are in general agreement with the contract. The effects of profile grade and camber would make exact verification difficult. Some differences in lengths, between top and bottom flange plates for example, are to be expected.

The procedures to follow in the event changes are required or requested by the fabricator can be found in Section 1.3.6 of this manual. In the past, shop plans with acceptable changes have been so noted and stamped.

> STRUCTURALLY ACCEPTABLE, BUT DOES NOT
> CONFORM TO THE CONTRACT REQUIREMENTS

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7.99 Bibliography

The following publications can provide general guidance for the design of steel structures. Some of this material may be dated and its application should be used with caution.

1. *U.S. Steel Highway Structures Design Handbook, Volumes I and II.*

   This is a detailed design reference for "I" girders and box girders, both straight and curved, utilizing either service load design or load factor design. Guidance for the design of wide flange beams is also included.

2. *Design of Welded Structures* by Omer H. Blodgett.

   This publication is quite helpful in the calculation of section properties and the design of individual members. There are sections on bridge girders and many other welded structures.

3. *Curved Girder Workshop* produced by the Federal Highway Administration.

   This publication is helpful in the design of curved "I" girders and box girders with explanation of the associated lateral flange bending, torsional, and warping stresses.

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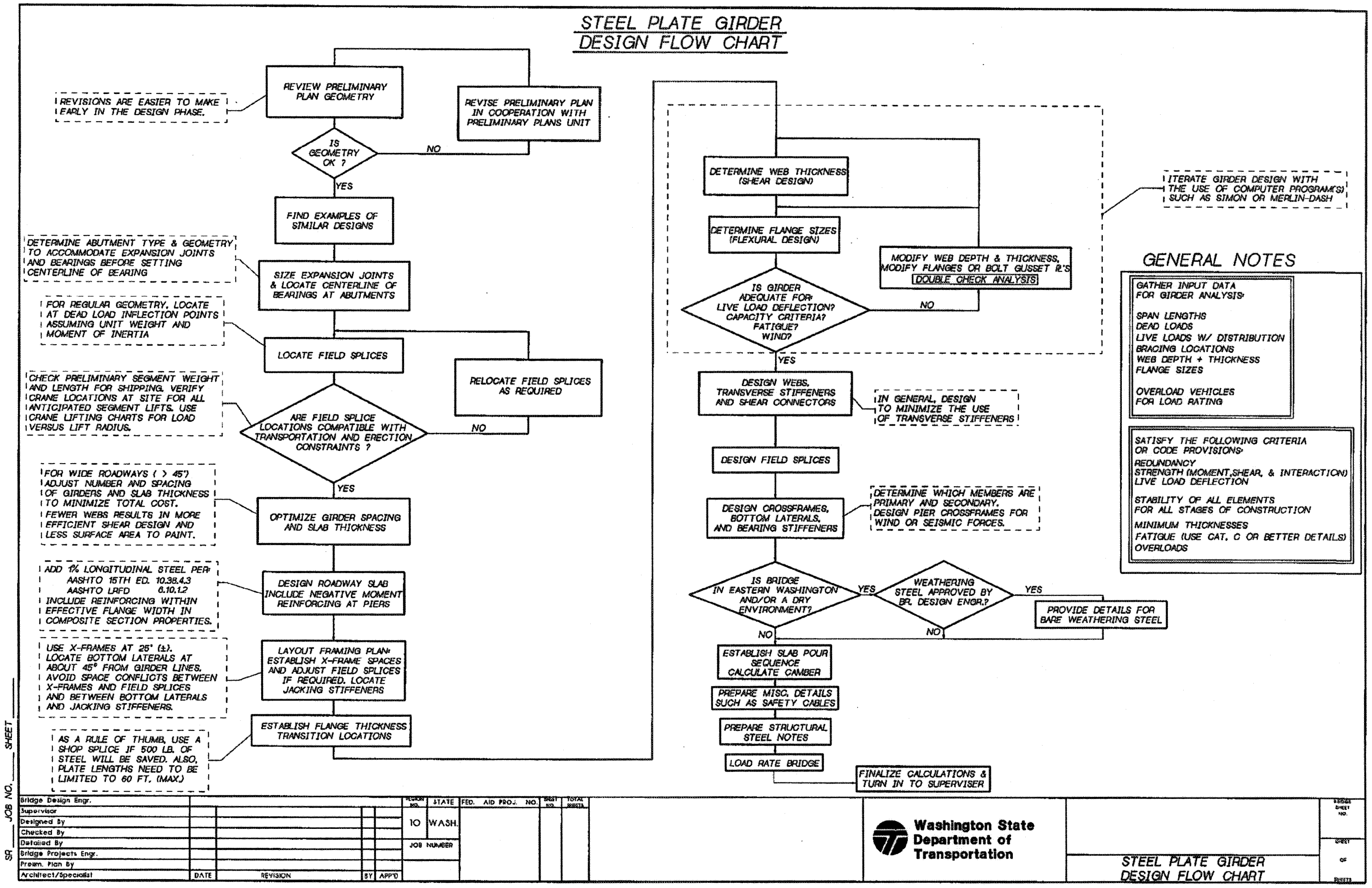
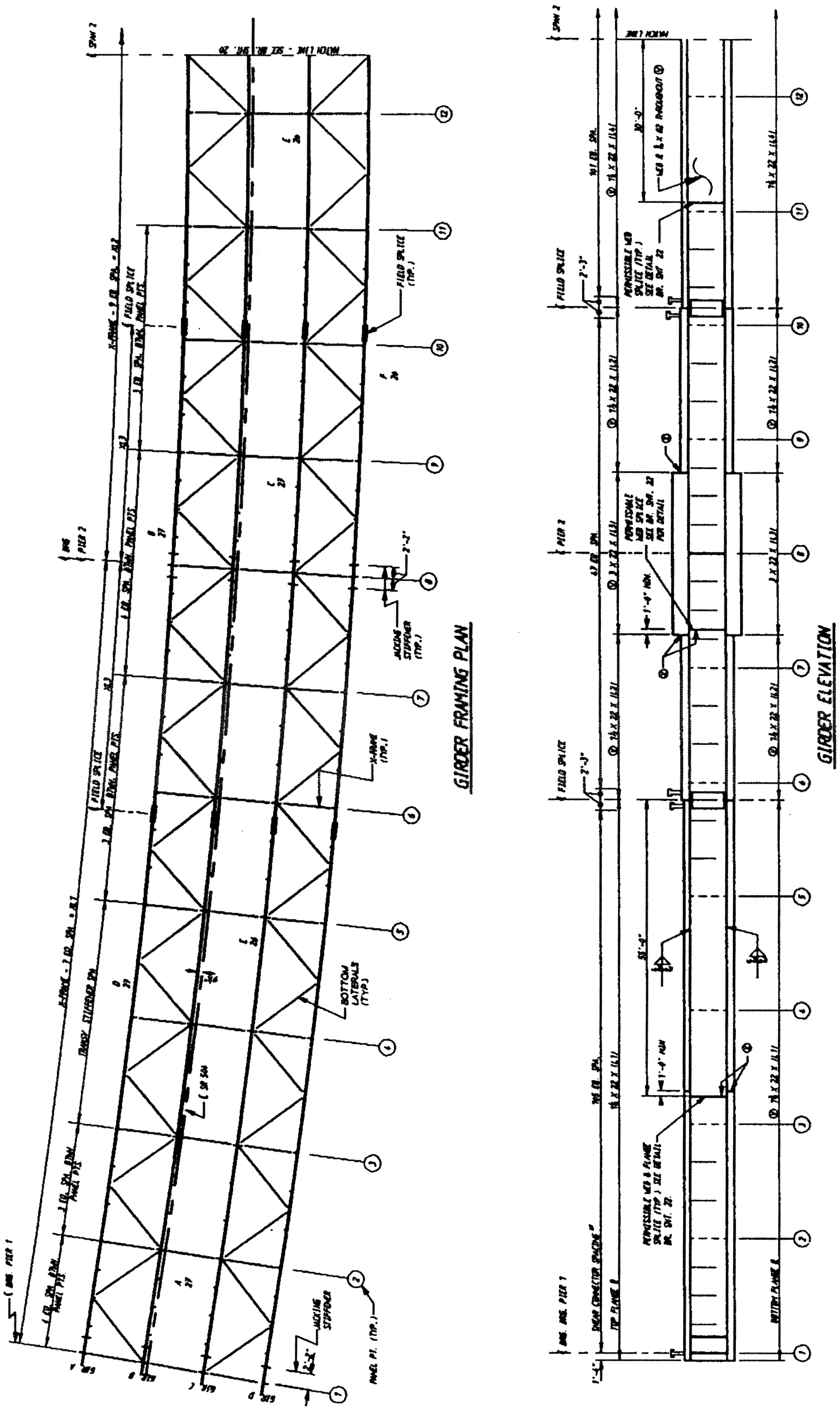
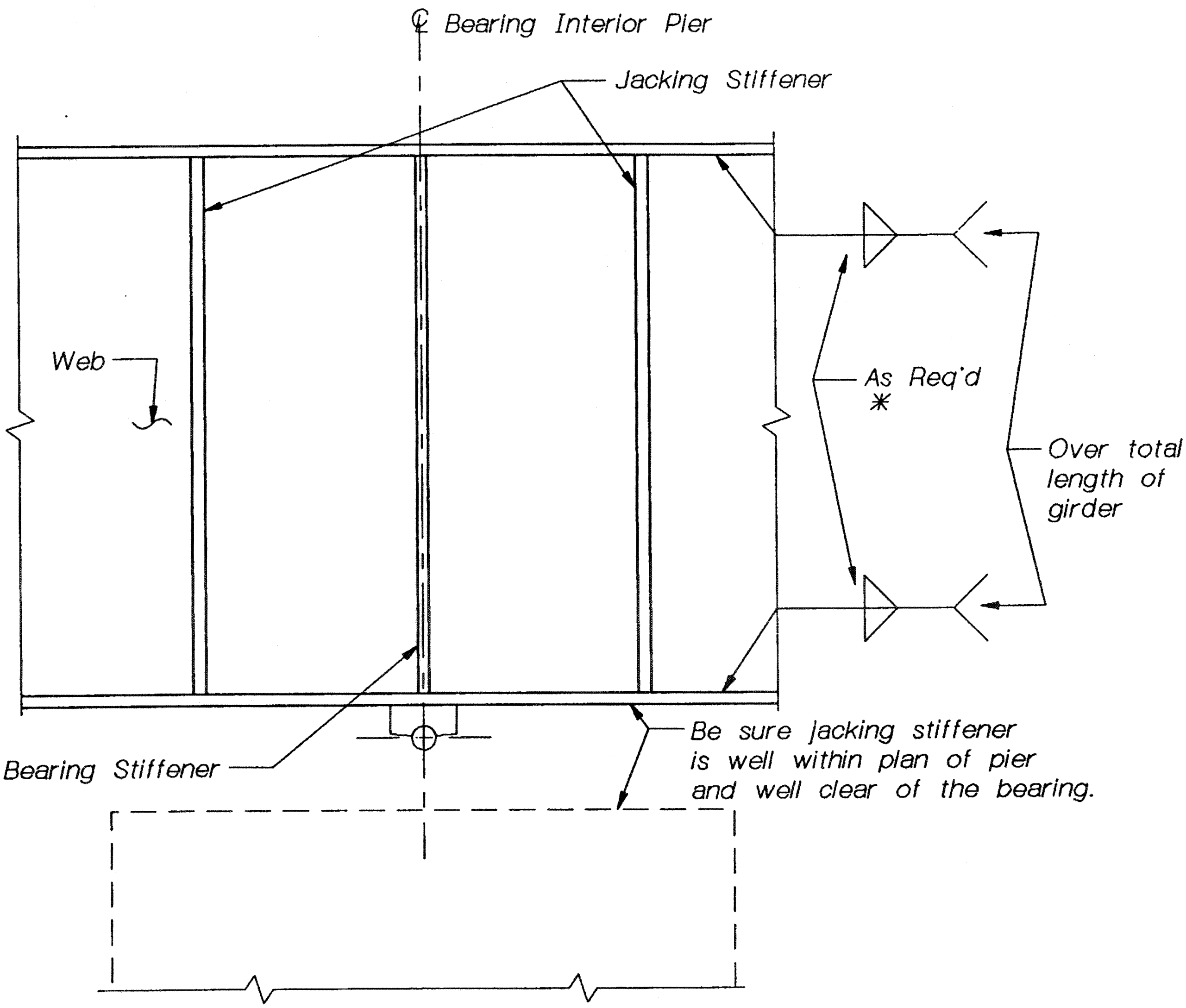
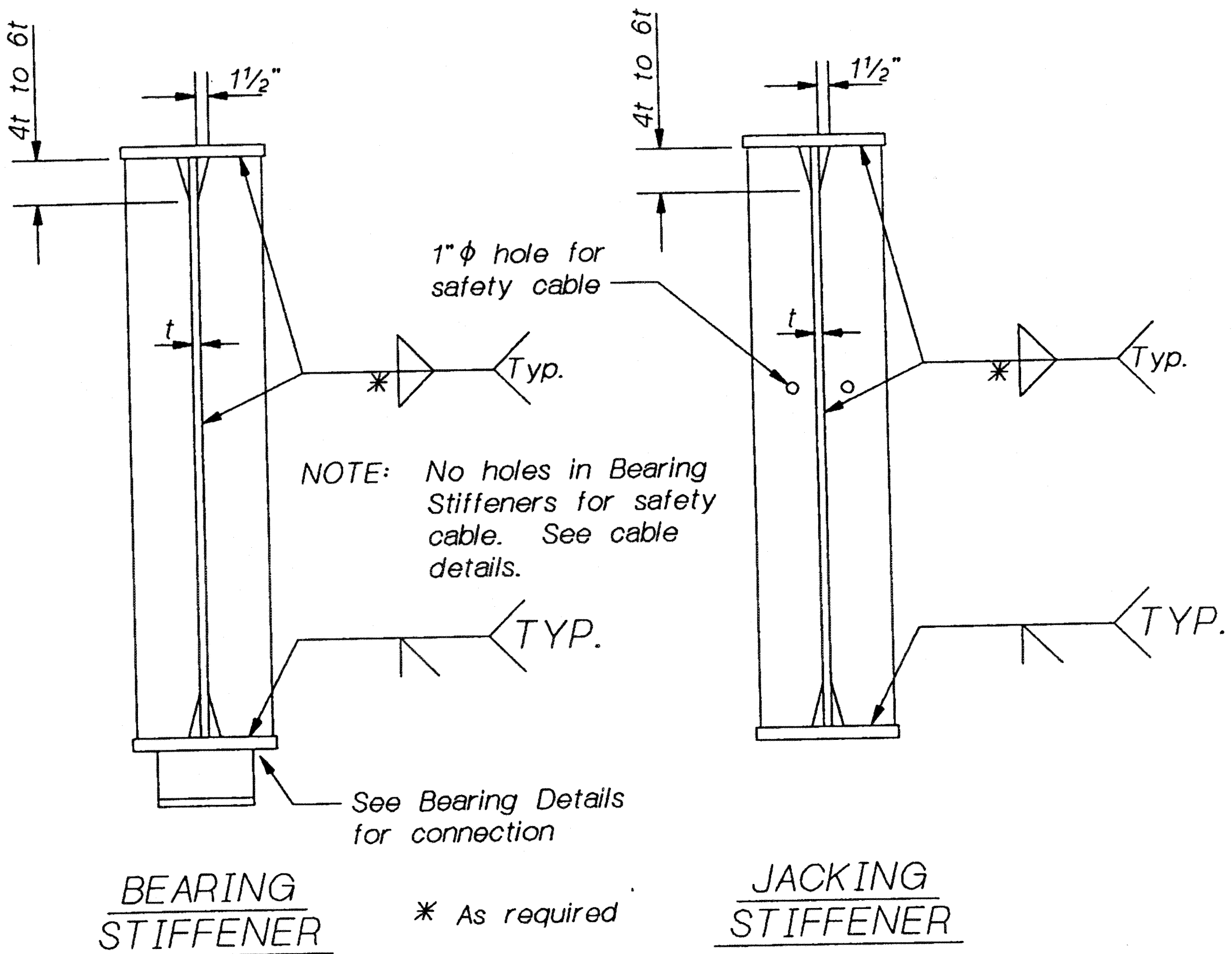

STEEL PLATE GIRDER
DESIGN FLOW CHART
REVISIONS ARE EASIER TO MAKE EARLY IN THE DESIGN PHASE.
REVIEW PRELIMINARY PLAN GEOMETRY
REVISE PRELIMINARY PLAN IN COOPERATION WITH PRELIMINARY PLANS UNIT
IS GEOMETRY OK ?
NO
YES
FIND EXAMPLES OF SIMILAR DESIGNS
DETERMINE ABUTMENT TYPE & GEOMETRY TO ACCOMMODATE EXPANSION JOINTS AND BEARINGS BEFORE SETTING CENTERLINE OF BEARING
SIZE EXPANSION JOINTS & LOCATE CENTERLINE OF BEARINGS AT ABUTMENTS
FOR REGULAR GEOMETRY, LOCATE AT DEAD LOAD INFLECTION POINTS ASSUMING UNIT WEIGHT AND MOMENT OF INERTIA
LOCATE FIELD SPLICES
RELOCATE FIELD SPLICES AS REQUIRED
CHECK PRELIMINARY SEGMENT WEIGHT AND LENGTH FOR SHIPPING. VERIFY CRANE LOCATIONS AT SITE FOR ALL ANTICIPATED SEGMENT LIFTS. USE CRANE LIFTING CHARTS FOR LOAD VERSUS LIFT RADIUS.
ARE FIELD SPLICE LOCATIONS COMPATIBLE WITH TRANSPORTATION AND ERECTION CONSTRAINTS ?
NO
YES
FOR WIDE ROADWAYS (> 45') ADJUST NUMBER AND SPACING OF GIRDERS AND SLAB THICKNESS TO MINIMIZE TOTAL COST. FEWER WEBS RESULTS IN MORE EFFICIENT SHEAR DESIGN AND LESS SURFACE AREA TO PAINT.
OPTIMIZE GIRDER SPACING AND SLAB THICKNESS
ADD 1% LONGITUDINAL STEEL PER: AASHTO 15TH ED. 10.38.4.3 AASHTO LRFD 6.10.1.2 INCLUDE REINFORCING WITHIN EFFECTIVE FLANGE WIDTH IN COMPOSITE SECTION PROPERTIES.
DESIGN ROADWAY SLAB INCLUDE NEGATIVE MOMENT REINFORCING AT PIERS
USE X-FRAMES AT 25' (±). LOCATE BOTTOM LATERALS AT ABOUT 45° FROM GIRDER LINES. AVOID SPACE CONFLICTS BETWEEN X-FRAMES AND FIELD SPLICES AND BETWEEN BOTTOM LATERALS AND JACKING STIFFENERS.
LAYOUT FRAMING PLAN: ESTABLISH X-FRAME SPACES AND ADJUST FIELD SPLICES IF REQUIRED. LOCATE JACKING STIFFENERS
AS A RULE OF THUMB, USE A SHOP SPLICE IF 500 LB. OF STEEL WILL BE SAVED. ALSO, PLATE LENGTHS NEED TO BE LIMITED TO 60 FT. (MAX.)
ESTABLISH FLANGE THICKNESS TRANSITION LOCATIONS
DETERMINE WEB THICKNESS (SHEAR DESIGN)
DETERMINE FLANGE SIZES (FLEXURAL DESIGN)
ITERATE GIRDER DESIGN WITH THE USE OF COMPUTER PROGRAM(S) SUCH AS SIMON OR MERLIN-DASH
MODIFY WEB DEPTH & THICKNESS, MODIFY FLANGES OR BOLT GUSSET ℞'S
DOUBLE CHECK ANALYSIS
IS GIRDER ADEQUATE FOR: LIVE LOAD DEFLECTION? CAPACITY CRITERIA? FATIGUE? WIND?
NO
YES
DESIGN WEBS, TRANSVERSE STIFFENERS AND SHEAR CONNECTORS
IN GENERAL, DESIGN TO MINIMIZE THE USE OF TRANSVERSE STIFFENERS
DESIGN FIELD SPLICES
DESIGN CROSSFRAMES, BOTTOM LATERALS, AND BEARING STIFFENERS
DETERMINE WHICH MEMBERS ARE PRIMARY AND SECONDARY. DESIGN PIER CROSSFRAMES FOR WIND OR SEISMIC FORCES.
IS BRIDGE IN EASTERN WASHINGTON AND/OR A DRY ENVIRONMENT?
YES
NO
WEATHERING STEEL APPROVED BY BR. DESIGN ENGR.?
YES
NO
PROVIDE DETAILS FOR BARE WEATHERING STEEL
ESTABLISH SLAB POUR SEQUENCE CALCULATE CAMBER
PREPARE MISC. DETAILS SUCH AS SAFETY CABLES
PREPARE STRUCTURAL STEEL NOTES
LOAD RATE BRIDGE
FINALIZE CALCULATIONS & TURN IN TO SUPERVISER
GENERAL NOTES
GATHER INPUT DATA FOR GIRDER ANALYSIS:
SPAN LENGTHS
DEAD LOADS
LIVE LOADS W/ DISTRIBUTION
BRACING LOCATIONS
WEB DEPTH + THICKNESS
FLANGE SIZES
OVERLOAD VEHICLES FOR LOAD RATING
SATISFY THE FOLLOWING CRITERIA OR CODE PROVISIONS:
REDUNDANCY
STRENGTH (MOMENT,SHEAR, & INTERACTION)
LIVE LOAD DEFLECTION
STABILITY OF ALL ELEMENTS FOR ALL STAGES OF CONSTRUCTION
MINIMUM THICKNESSES
FATIGUE (USE CAT. C OR BETTER DETAILS)
OVERLOADS
Bridge Design Engr.
Supervisor
Designed By
Checked By
Detailed By
Bridge Projects Engr.
Prelim. Plan By
Architect/Specialist
DATE
REVISION
BY
APP'D
STATE
FED. AID PROJ. NO.
10
WASH.
JOB NUMBER
Washington State Department of Transportation
STEEL PLATE GIRDER DESIGN FLOW CHART
SR
JOB NO.
SHEET

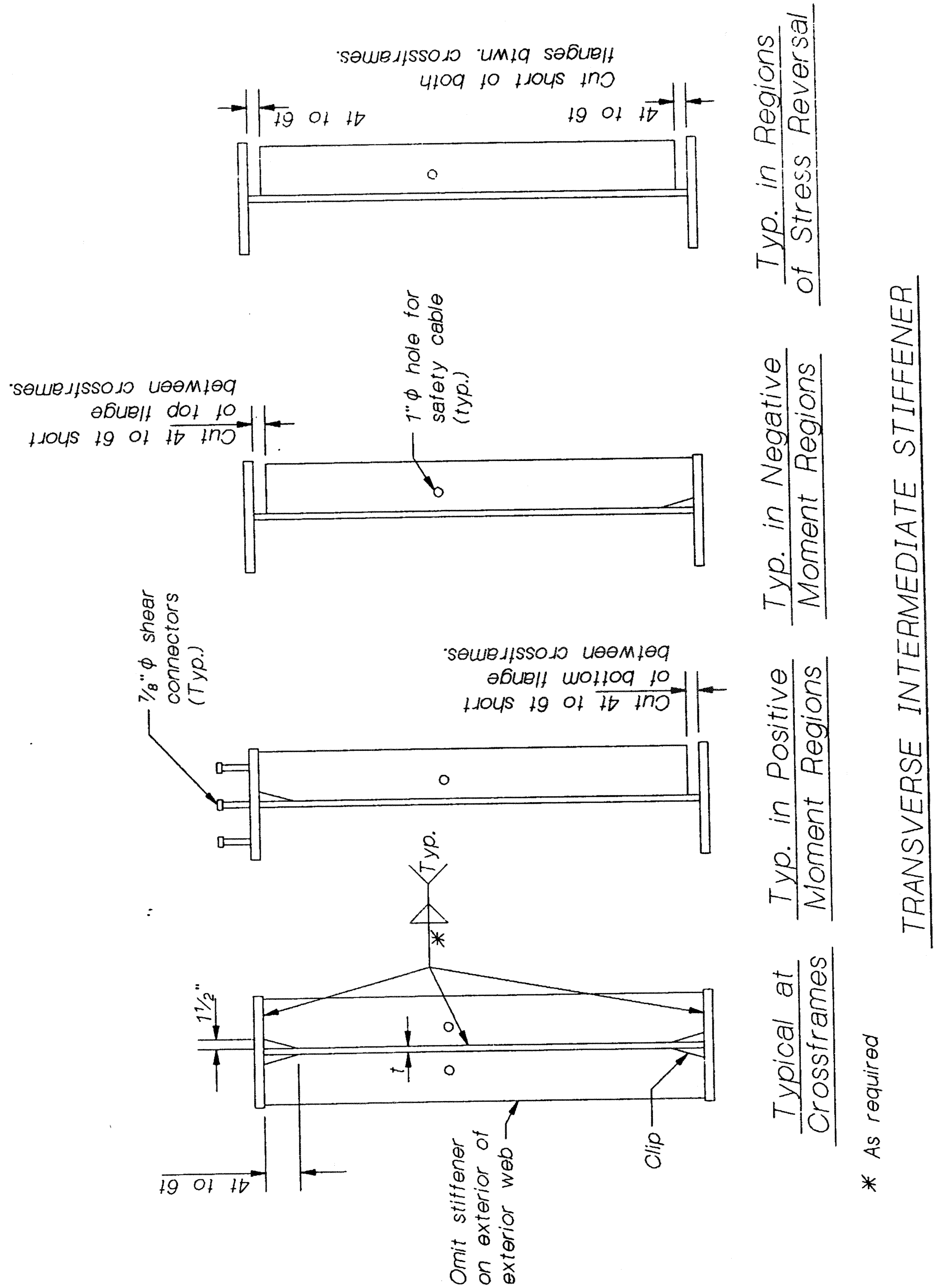

GIRDER FRAMING PLAN
GIRDER ELEVATION

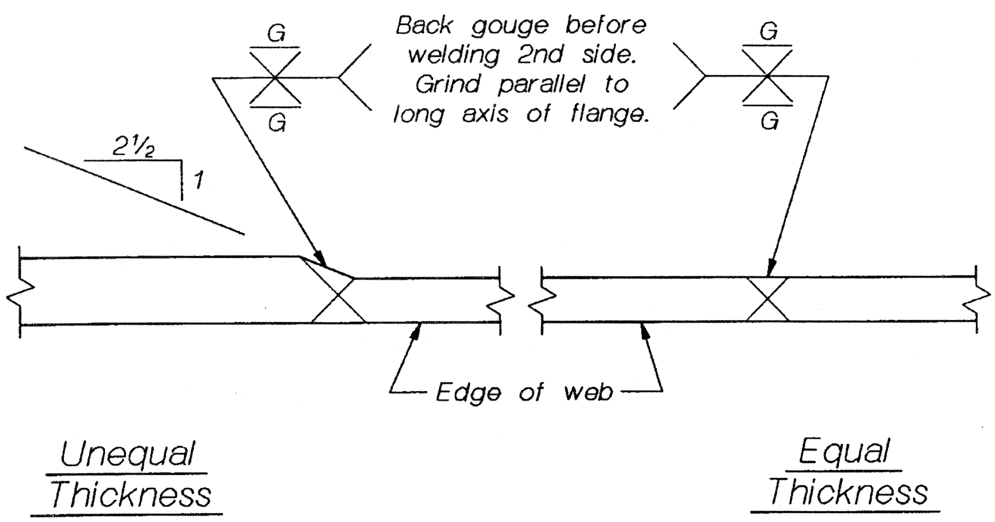


Part Longitudinal Girder Elevation

* Strive to keep this weld size to 5/16 or less. Bridge Welding Code limits single pass fillet weld size to 5/16" for most popular processes.

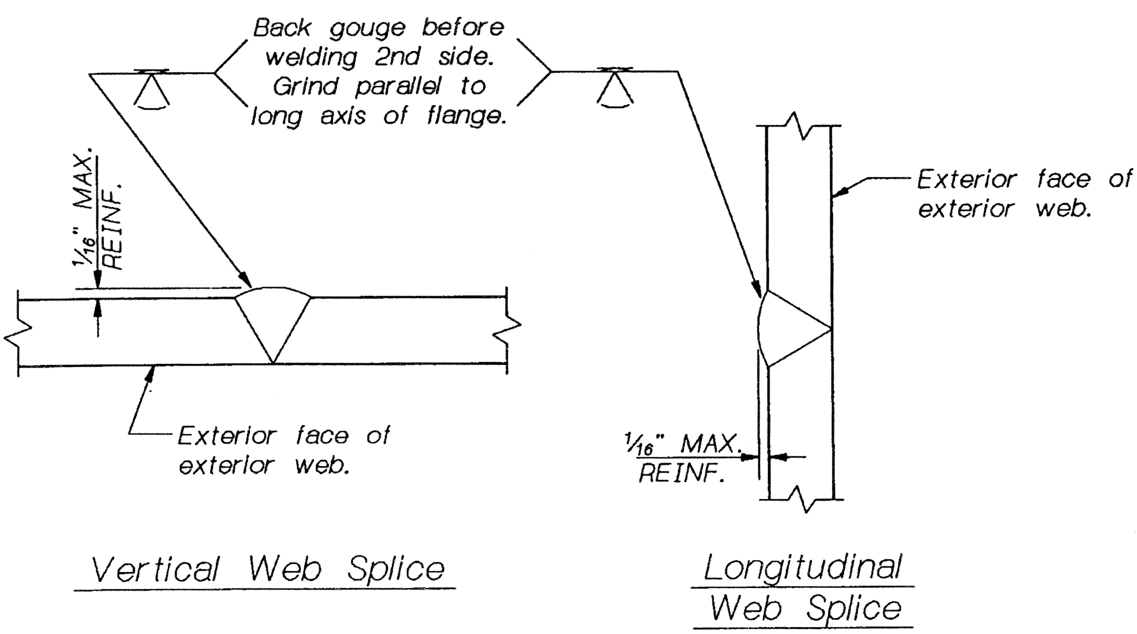

4t to 6t
1½"
4t to 6t
1½"
1" ϕ hole for safety cable
t
t
✱
Typ.
✱
Typ.
NOTE: No holes in Bearing Stiffeners for safety cable. See cable details.
TYP.
TYP.
See Bearing Details for connection
BEARING STIFFENER
✱ As required
JACKING STIFFENER

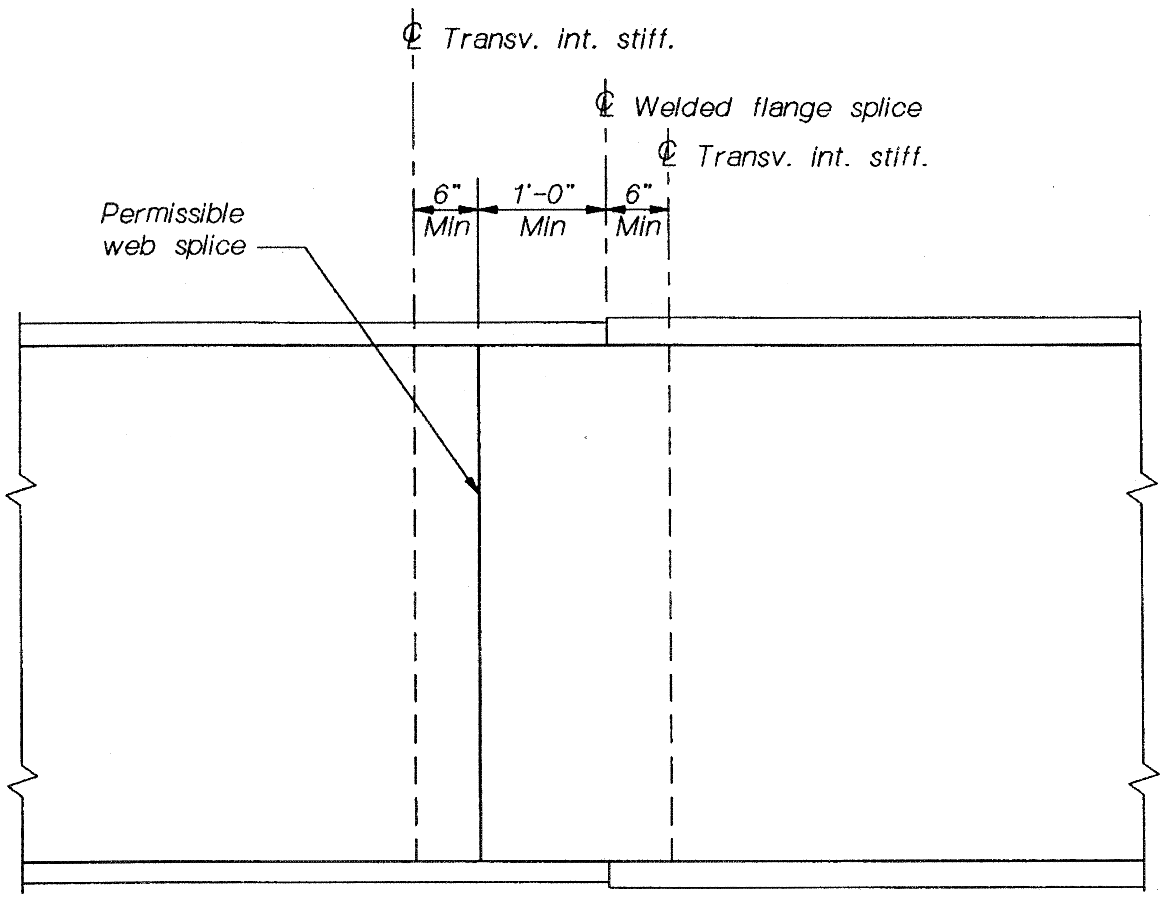

TRANSVERSE INTERMEDIATE STIFFENER
Typical at Crossframes
Typ. in Positive Moment Regions
Typ. in Negative Moment Regions
Typ. in Regions of Stress Reversal
4t to 6t
Cut 4t to 6t short of bottom flange between crossframes.
Cut 4t to 6t short of top flange between crossframes.
Cut short of both flanges btwn. crossframes.
7/8" φ shear connectors (Typ.)
1" φ hole for safety cable (typ.)
1½"
t
Typ.
Clip
Omit stiffener on exterior of exterior web
✱ As required



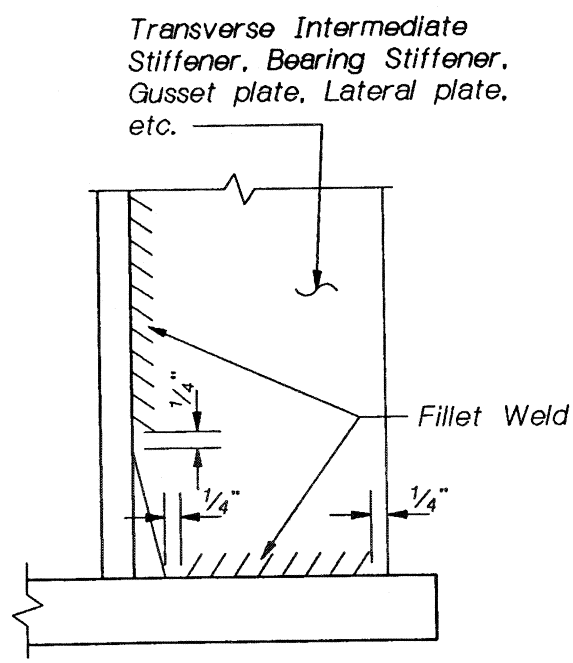
TENSION OR COMPRESSION FLANGE SPLICE

Figure 7.4.5–3

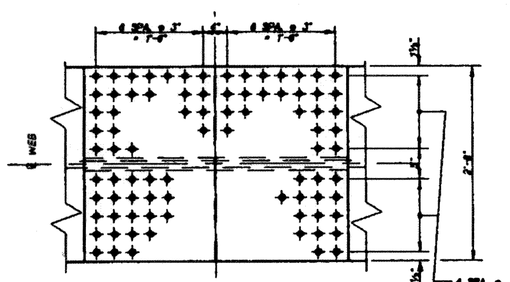




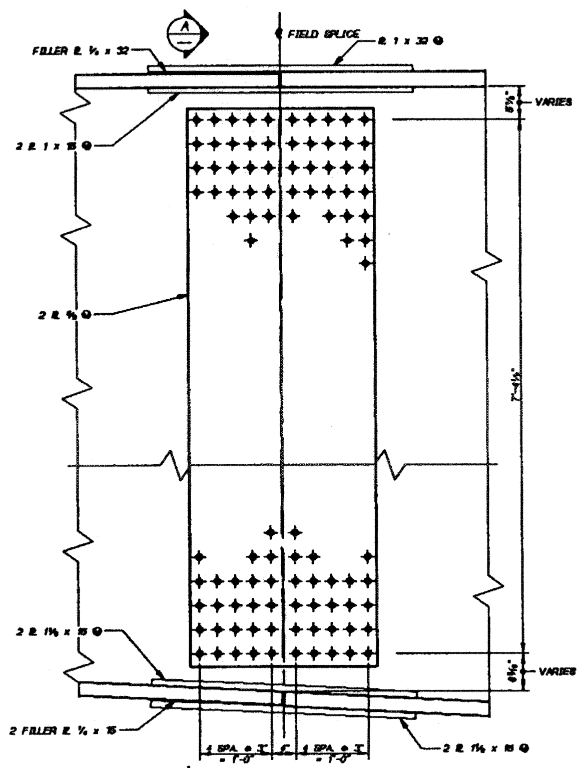
OPTIONAL WEB SPLICE



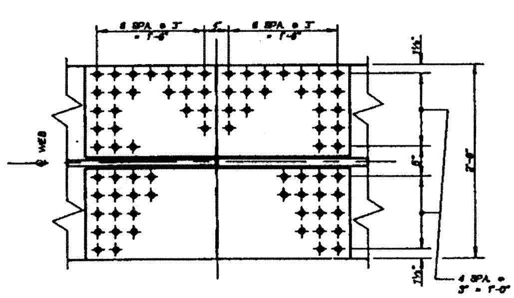
FILLET WELD TERMINATION DETAIL



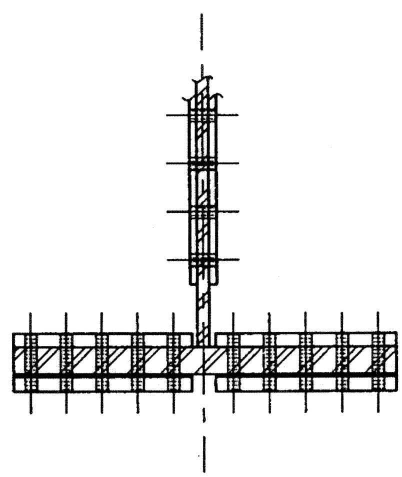
PLAN
TOP FLANGE



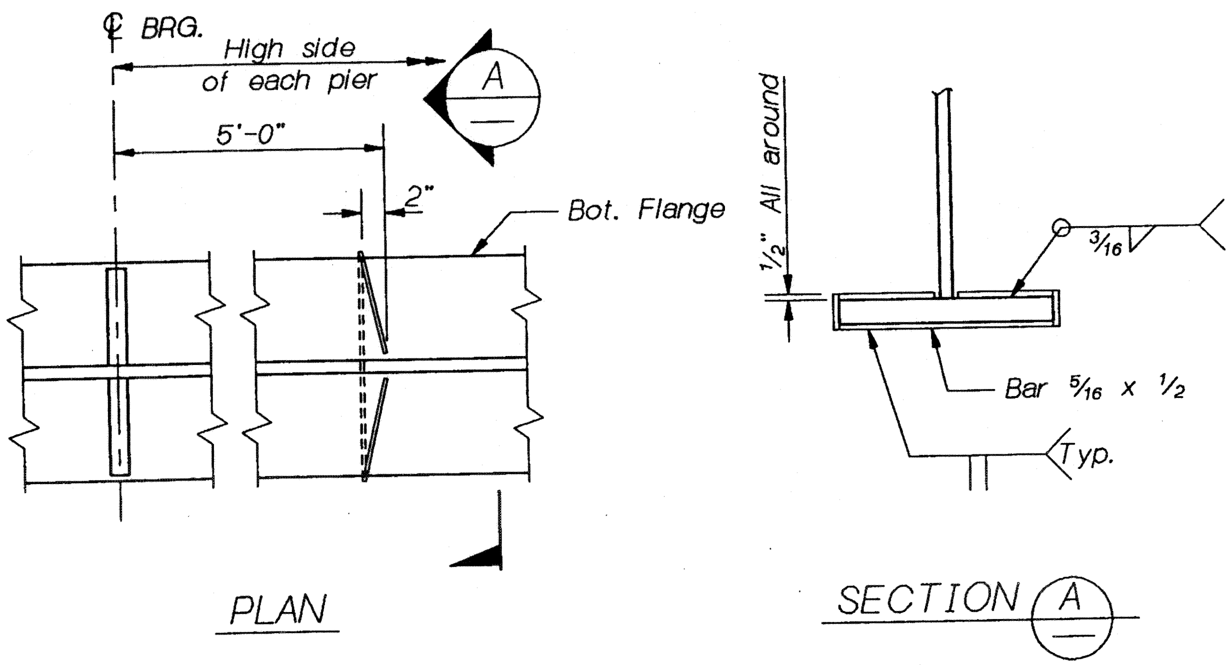
ELEVATION



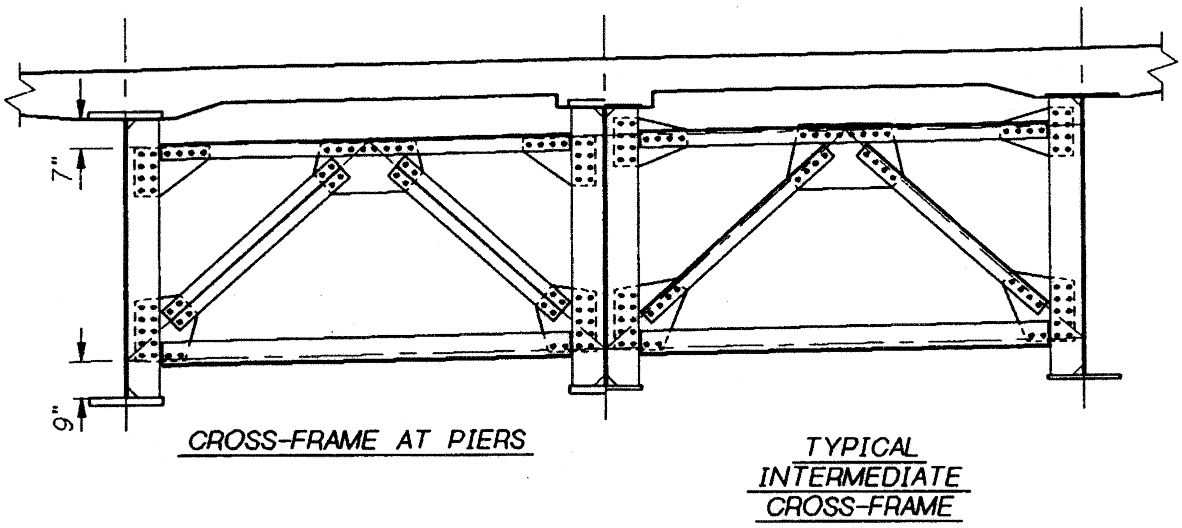
PLAN
BOTTOM FLANGE

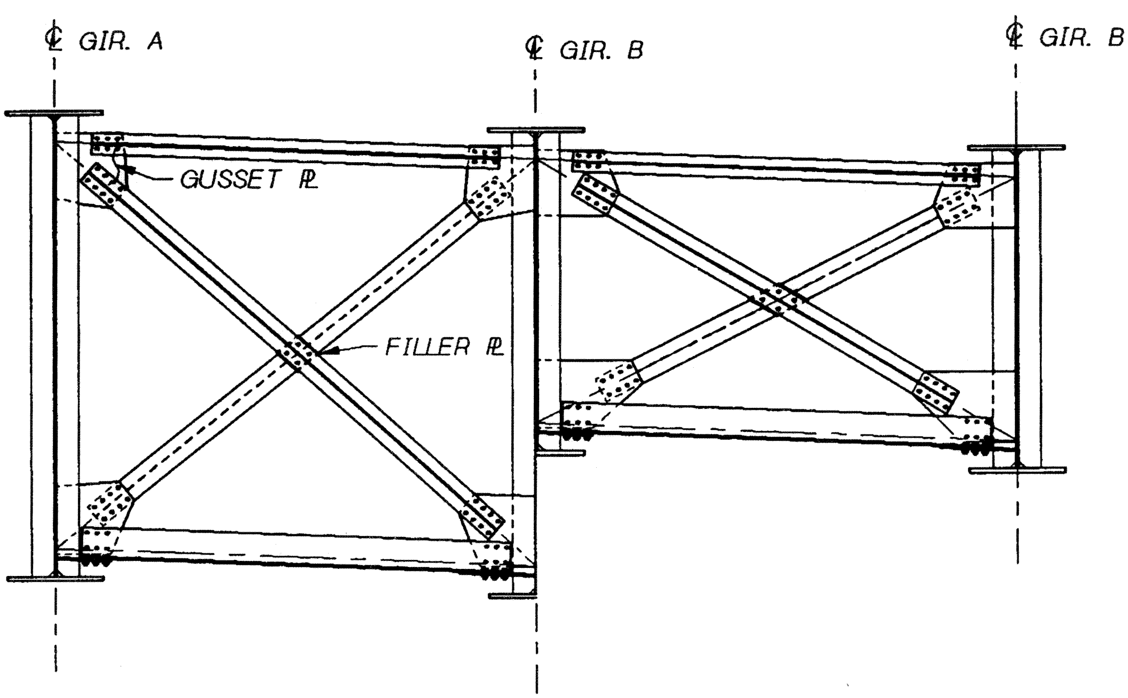


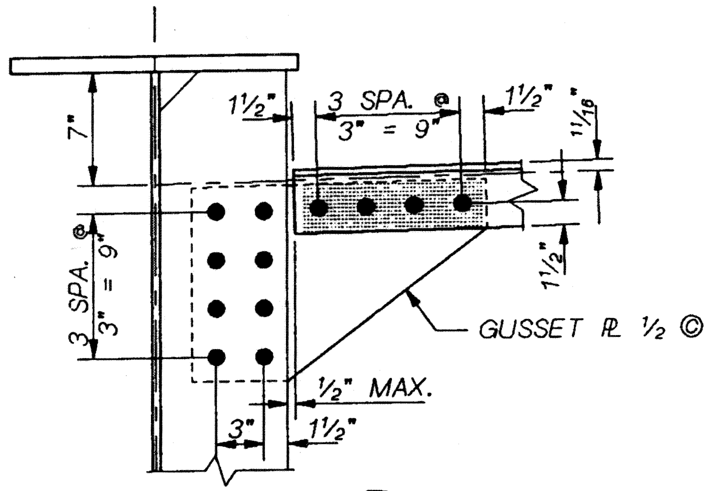
SECTION A



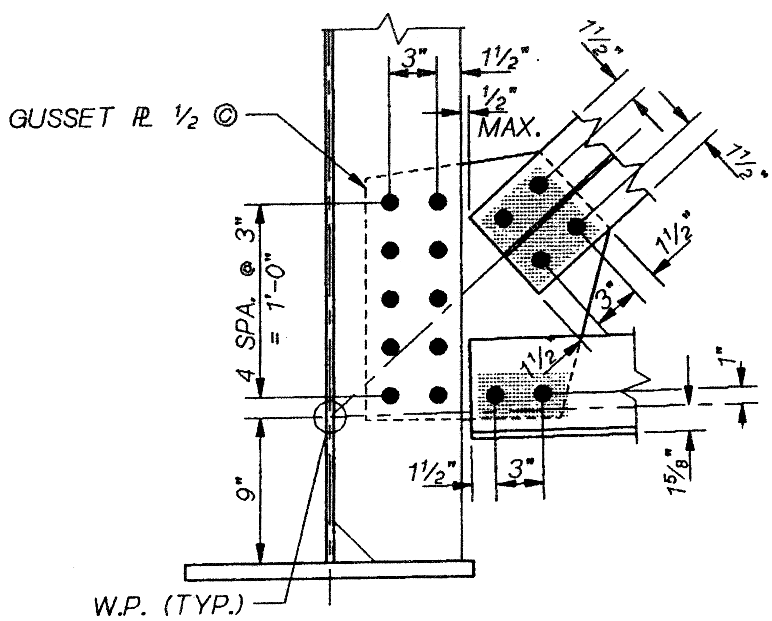
DRIP PLATE DETAILS
Exterior Girder


7"
9"
CROSS-FRAME AT PIERS
TYPICAL
INTERMEDIATE
CROSS-FRAME


℄ GIR. A
℄ GIR. B
℄ GIR. B
GUSSET ℙ
FILLER ℙ



DETAIL A



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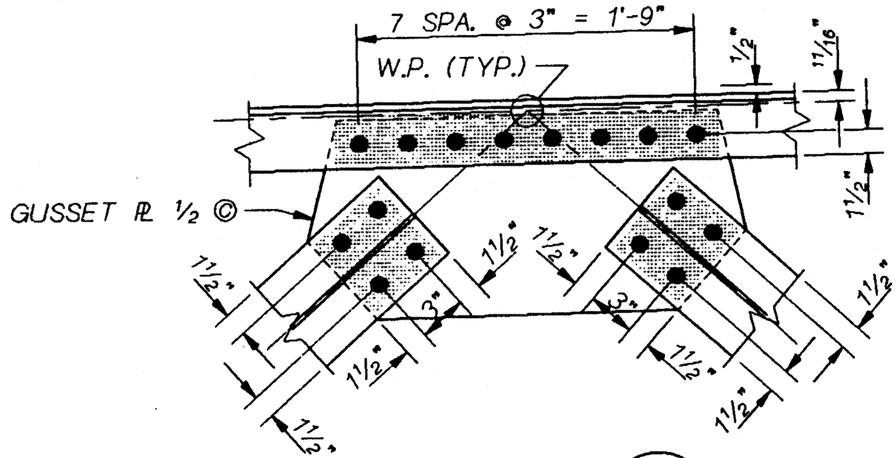

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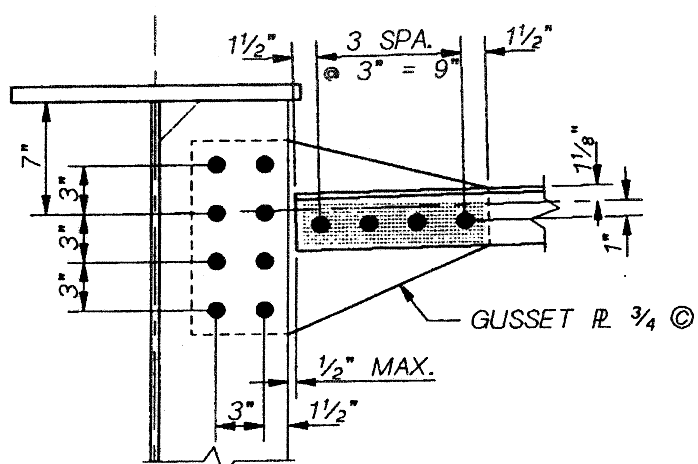

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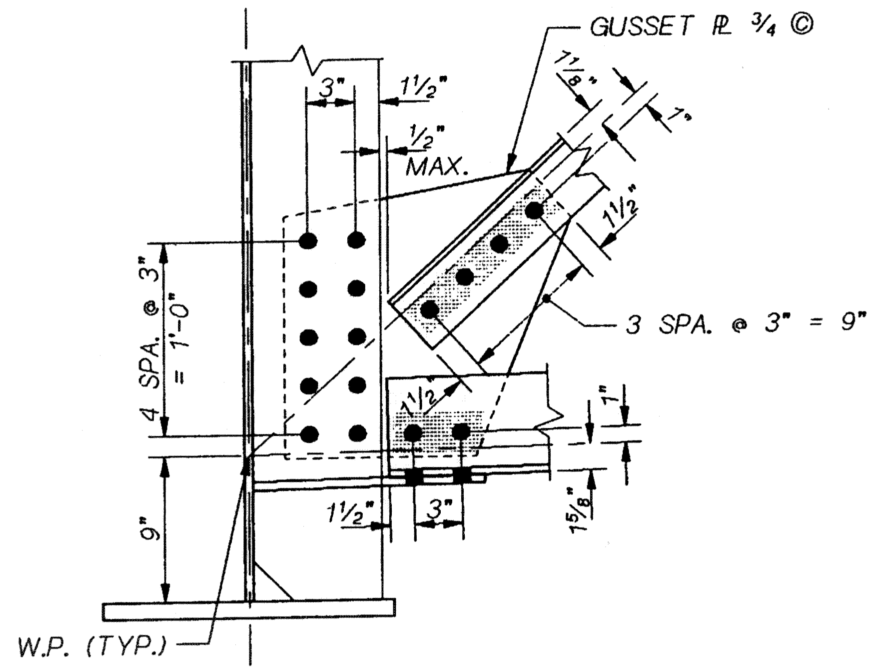

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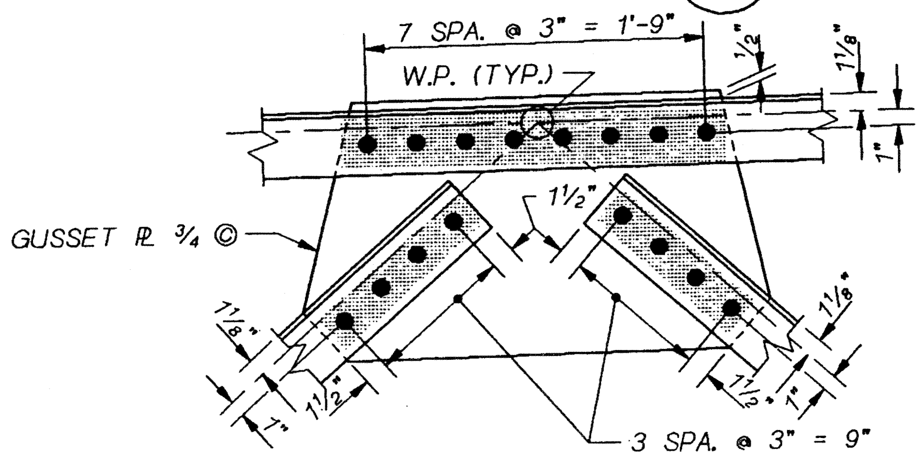
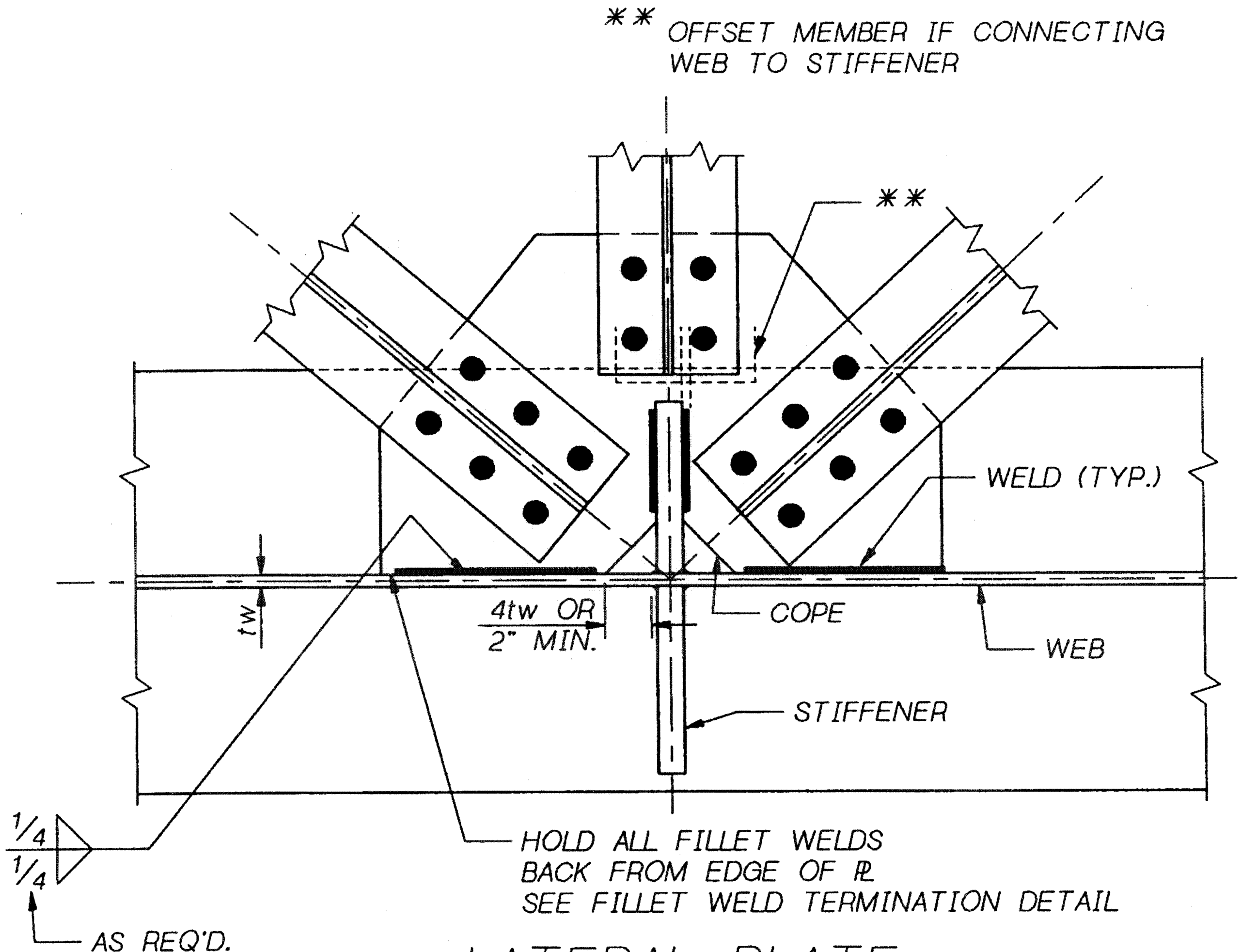
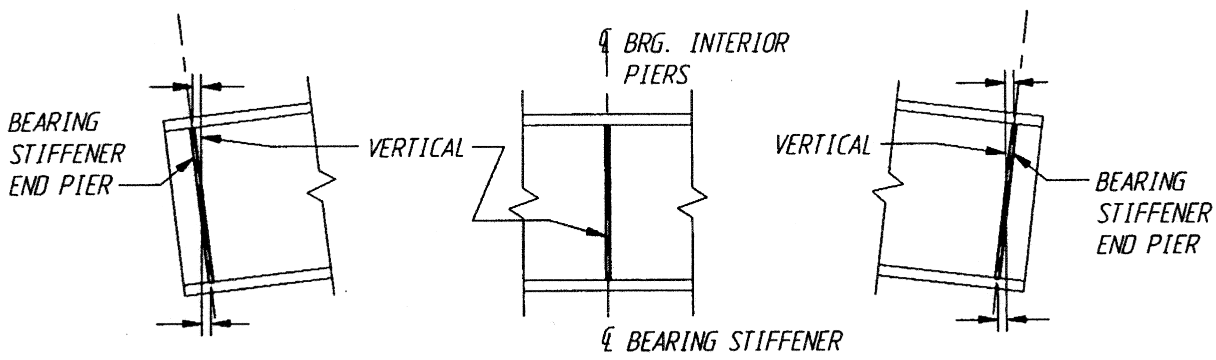


DETAIL F



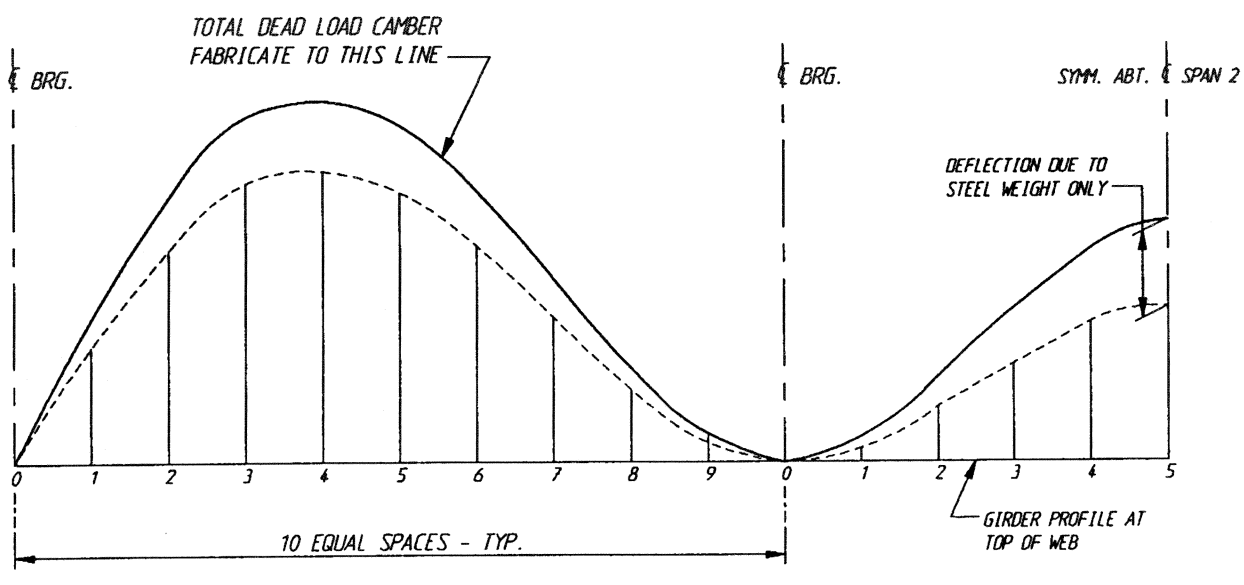
LATERAL PLATE DETAIL

NOTE: CHECK CAT. E STRESS RANGE FOR THIS DETAIL.



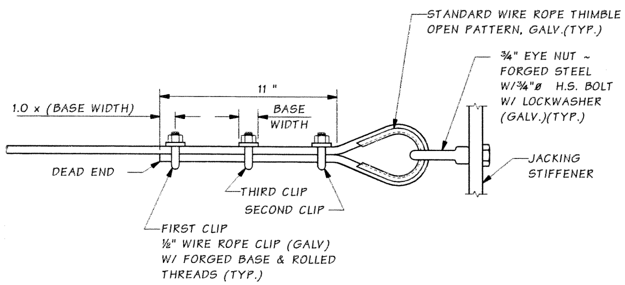
BEARING STIFFENER CAMBER DIAGRAM

DIAGRAM DOES NOT INCLUDE EFFECTS OF PROFILE GRADE.

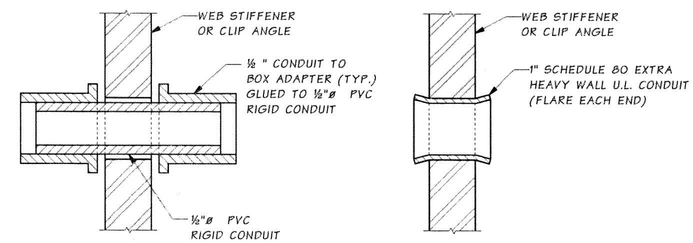


DEAD LOAD CAMBER DIAGRAM

TENTH POINTS MEASURED ALONG ℄ OF GIRDER
+ DENOTES DIMENSIONS LESS THAN 1/16"
INCLUDES EFFECTS OF SLAB SHRINKAGE



WIRE ROPE CLIP

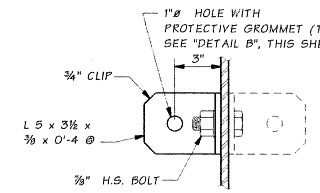


DETAIL B
OPTION 1

DETAIL B
OPTION 2

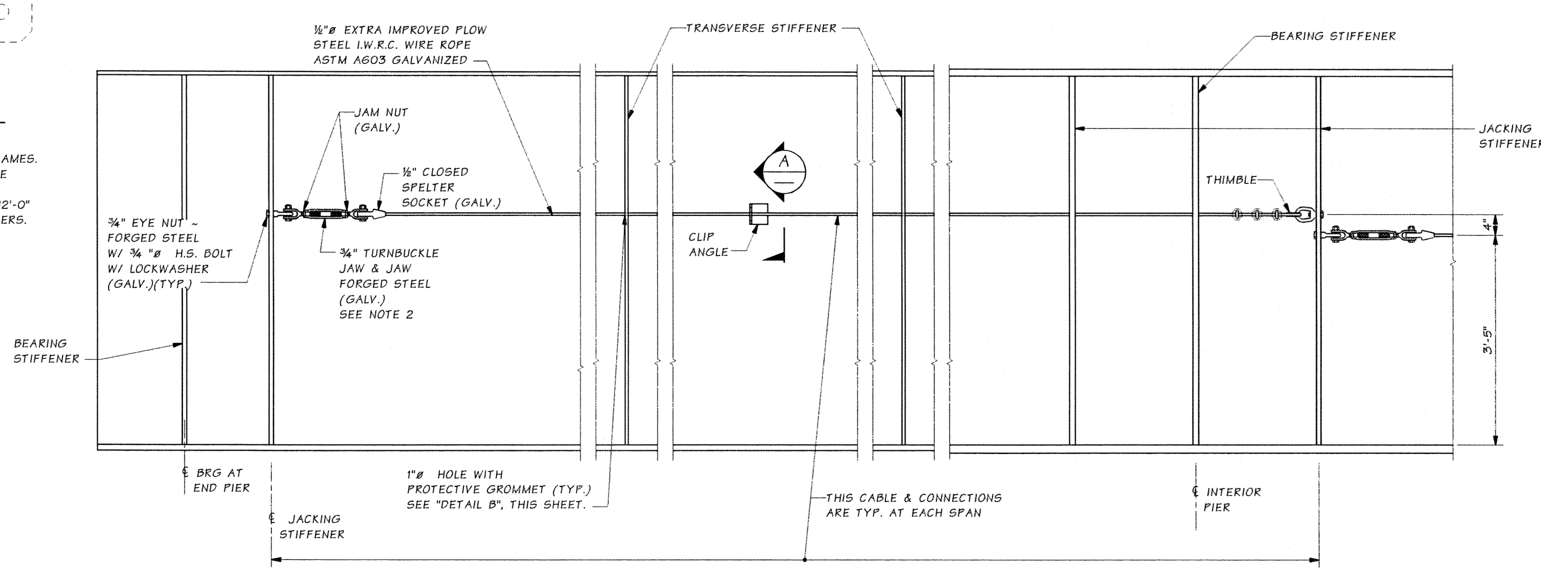
NOTES:

1. THE SAFETY CABLE SHALL BE STRETCHED TAUT TO MINIMIZE DEFLECTION.
2. TURNBUCKLE TAKE UP BODY LENGTH AS REQUIRED TO PRODUCE TAUT SAFETY CABLE CONDITION.
3. TURN BACK ROPE TO 11½ INCHES FROM THE THIMBLE.
4. ATTACH FIRST CLIP ONE BASE WIDTH FROM DEAD END OF ROPE. CAUTION: U-BOLT OVER DEAD END OF ROPE-SADDLE OR BASE BEARS ON LIVE END. TIGHTEN NUTS TO RECOMMENDED TORQUE.
5. ATTACH SECOND CLIP AS CLOSE TO THIMBLE AS POSSIBLE. TIGHTEN NUTS FIRMLY, BUT NOT COMPLETELY TIGHT.
6. ATTACH THIRD CLIP EQUALLY SPACED BETWEEN THE TWO CLIPS APPLIED PREVIOUSLY. TIGHTEN NUTS - TAKE UP ANY ROPE SLACK - UNIFORMLY TIGHTEN ALL NUTS TO 65 FOOT POUND TORQUE.
7. CHECK AND TIGHTEN CLIP ATTACHMENTS REGULARLY TO COMPENSATE FOR ROPE DIAMETER REDUCTION OR POSSIBLE SLIPPAGE.
8. CABLES SHALL ALTERNATE BETWEEN 3'-9" AND 3'-5" FROM THE BOTTOM FLANGE.



SECTION A

USE CLIP ANGLES MIDWAY BETWEEN X-FRAMES. OMIT WHERE STIFFENERS EXIST OR WHERE X-FRAME SPACING IS LESS THAN 12'-0". PLACE CLIP ANGLES AT APPROXIMATELY 12'-0" O.C. ON OUTSIDE FACE OF EXTERIOR GIRDERS.



ELEVATION
CABLES REQUIRED ON BOTH SIDES OF ALL GIRDERS.

Bridge Design Engr.					REGION NO.	STATE	FED. AID PROJ. NO.	SHEET NO. / TOTAL SHEETS
Supervisor					10	WASH.		
Designed By					JOB NUMBER			
Checked By								
Detailed By								
Bridge Projects Engr.								
Prelim. Plan By								
Architect/Specialist	DATE	REVISION	BY	APP'D				

Wed Jul 03 13:16:04 2002

BRIDGE AND STRUCTURES OFFICE

Washington State Department of Transportation

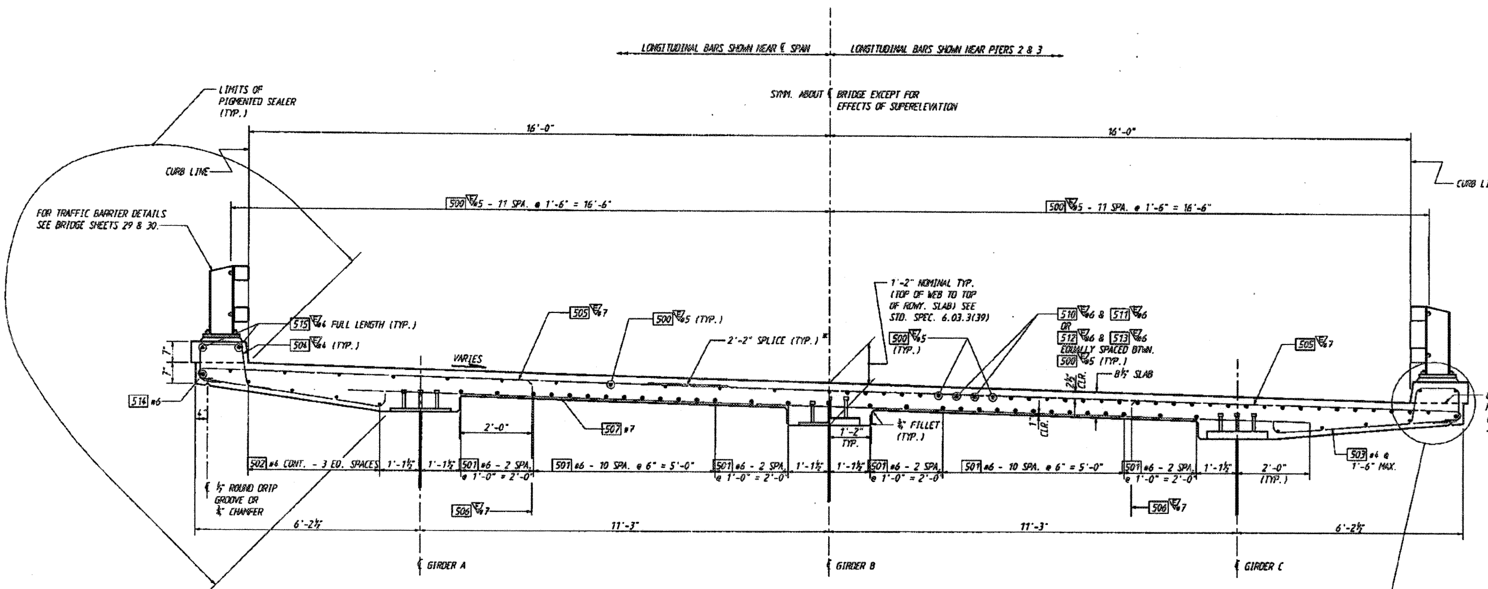
STANDARD SAFETY CABLES

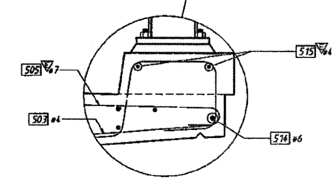
SAFETY CABLE

BRIDGE SHEET NO.

SHEET OF SHEETS

SR JOB NO. SHEET





TYPICAL ROADWAY SECTION

* ALTERNATE SPLICES ABOUT ℄ BRIDGE

NOTES:

ALL LONGITUDINAL DIMENSIONS ARE MEASURED ALONG ℄ BRIDGE EXCEPT AS NOTED.

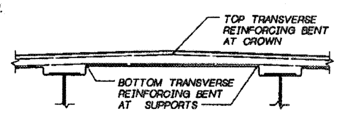
BAR LENGTHS & SPLICE LOCATIONS NOT SHOWN FOR LONGITUDINAL REINFORCEMENT AT CONTRACTOR'S OPTION.

SPLICE LENGTH IN LONGITUDINAL REINFORCEMENT SHALL BE 2'-0" MIN.

SPLICES NOT ALLOWED WITHIN 25'-0" OF ℄ PIERS 2 & 3 EXCEPT AS NOTED.

NO MORE THAN HALF OF THE CONTINUOUS BARS SHALL BE SPLICED AT ONE LOCATION.

▽ DENOTES EPOXY COATED BARS.



DETAIL OF BOTTOM OF SLAB AT CROWN POINT

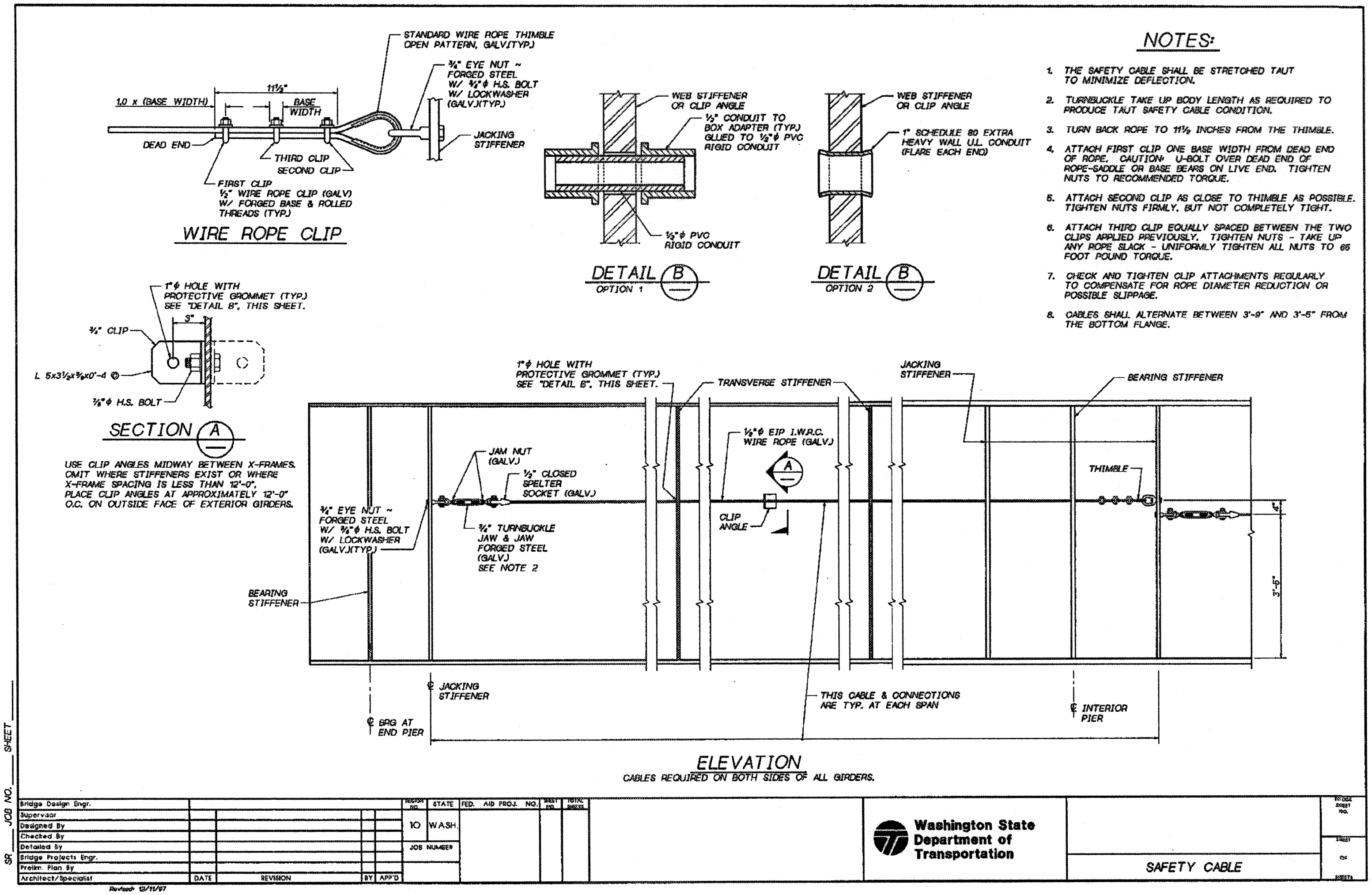
Bridge Design Engr.					REGION NO.	STATE	FED. AID PROJ. NO.	SHEET NO.	TOTAL SHEETS
Supervisor					10	WASH.			
Designed By					JOB NUMBER				
Checked By									
Detailed By									
Bridge Projects Engr.									
Prelim. Plan By									
Architect/Specialist	DATE	REVISION	BY	APP'D					

SR ___ JOB NO. ___ SHEET ___

Washington State Department of Transportation

TYPICAL SECTION

BRIDGE SHEET NO. ___ SHEET ___ OF ___ SHEETS


STANDARD WIRE ROPE THIMBLE OPEN PATTERN, GALV.(TYP.)
¾" EYE NUT ~ FORGED STEEL W/ ¾"ϕ H.S. BOLT W/ LOCKWASHER (GALV.)(TYP.)
1.0 x (BASE WIDTH)
11½"
BASE WIDTH
DEAD END
JACKING STIFFENER
THIRD CLIP
SECOND CLIP
FIRST CLIP ½" WIRE ROPE CLIP (GALV) W/ FORGED BASE & ROLLED THREADS (TYP.)
WIRE ROPE CLIP
WEB STIFFENER OR CLIP ANGLE
½" CONDUIT TO BOX ADAPTER (TYP.) GLUED TO ½"ϕ PVC RIGID CONDUIT
½"ϕ PVC RIGID CONDUIT
DETAIL B OPTION 1
WEB STIFFENER OR CLIP ANGLE
1" SCHEDULE 80 EXTRA HEAVY WALL U.L. CONDUIT (FLARE EACH END)
DETAIL B OPTION 2
NOTES:
1. THE SAFETY CABLE SHALL BE STRETCHED TAUT TO MINIMIZE DEFLECTION.
2. TURNBUCKLE TAKE UP BODY LENGTH AS REQUIRED TO PRODUCE TAUT SAFETY CABLE CONDITION.
3. TURN BACK ROPE TO 11½ INCHES FROM THE THIMBLE.
4. ATTACH FIRST CLIP ONE BASE WIDTH FROM DEAD END OF ROPE. CAUTION: U-BOLT OVER DEAD END OF ROPE-SADDLE OR BASE BEARS ON LIVE END. TIGHTEN NUTS TO RECOMMENDED TORQUE.
5. ATTACH SECOND CLIP AS CLOSE TO THIMBLE AS POSSIBLE. TIGHTEN NUTS FIRMLY, BUT NOT COMPLETELY TIGHT.
6. ATTACH THIRD CLIP EQUALLY SPACED BETWEEN THE TWO CLIPS APPLIED PREVIOUSLY. TIGHTEN NUTS - TAKE UP ANY ROPE SLACK - UNIFORMLY TIGHTEN ALL NUTS TO 65 FOOT POUND TORQUE.
7. CHECK AND TIGHTEN CLIP ATTACHMENTS REGULARLY TO COMPENSATE FOR ROPE DIAMETER REDUCTION OR POSSIBLE SLIPPAGE.
8. CABLES SHALL ALTERNATE BETWEEN 3'-9" AND 3'-5" FROM THE BOTTOM FLANGE.
1"ϕ HOLE WITH PROTECTIVE GROMMET (TYP.) SEE "DETAIL B", THIS SHEET.
¾" CLIP
3"
L 5x3½x⅜x0'-4 ©
⅞"ϕ H.S. BOLT
SECTION A
USE CLIP ANGLES MIDWAY BETWEEN X-FRAMES. OMIT WHERE STIFFENERS EXIST OR WHERE X-FRAME SPACING IS LESS THAN 12'-0". PLACE CLIP ANGLES AT APPROXIMATELY 12'-0" O.C. ON OUTSIDE FACE OF EXTERIOR GIRDERS.
1"ϕ HOLE WITH PROTECTIVE GROMMET (TYP.) SEE "DETAIL B", THIS SHEET.
TRANSVERSE STIFFENER
JACKING STIFFENER
BEARING STIFFENER
JAM NUT (GALV.)
½" CLOSED SPELTER SOCKET (GALV.)
½"ϕ EIP I.W.R.C. WIRE ROPE (GALV.)
THIMBLE
¾" EYE NUT ~ FORGED STEEL W/ ¾"ϕ H.S. BOLT W/ LOCKWASHER (GALV.)(TYP.)
¾" TURNBUCKLE JAW & JAW FORGED STEEL (GALV.) SEE NOTE 2
CLIP ANGLE
4"
3'-5"
BEARING STIFFENER
℄ JACKING STIFFENER
℄ BRG AT END PIER
THIS CABLE & CONNECTIONS ARE TYP. AT EACH SPAN
℄ INTERIOR PIER
ELEVATION
CABLES REQUIRED ON BOTH SIDES OF ALL GIRDERS.
Bridge Design Engr.
Supervisor
Designed By
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Detailed By
Bridge Projects Engr.
Prelim. Plan By
Architect/Specialist
DATE
REVISION
BY
APP'D
Revised: 12/11/97
STATE
FED. AID PROJ. NO.
10
WASH.
JOB NUMBER
Washington State Department of Transportation
SAFETY CABLE
SR
JOB NO.
SHEET